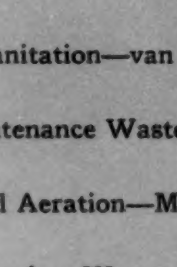
Journal

WATER POLLUTION CONTROL FEDERATION

FORMERLY SEWAGE AND INDUSTRIAL WASTES



Special Features

Space Sanitation—van der Wal

Aircraft Maintenance Wastes—Reid et al.

Surfactants and Aeration—Mancy and Okun

Heat Loss from Surface Water—Velz and Gannon

Aerated Lagoons—O'Connor and Eckenfelder

Operator's Corner

Sewer Maintenance, Air Pollution, Comminutors

APRIL 1960

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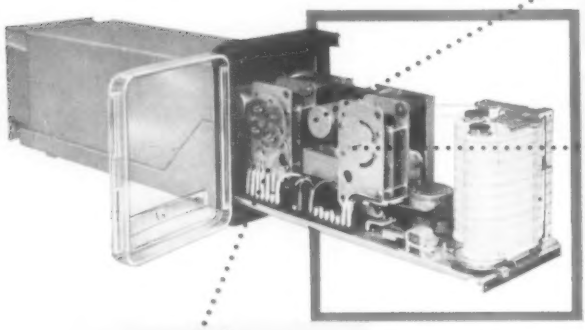

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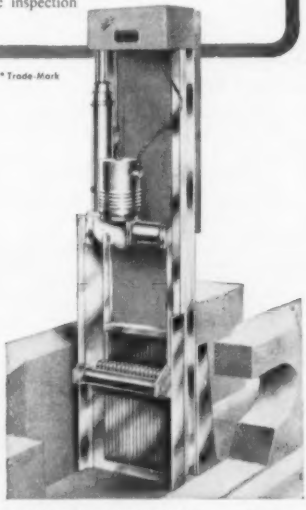

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JOURNAL of the WATER POLLUTION CONTROL FEDERATION

Volume 32 | April 1960 | Number 4

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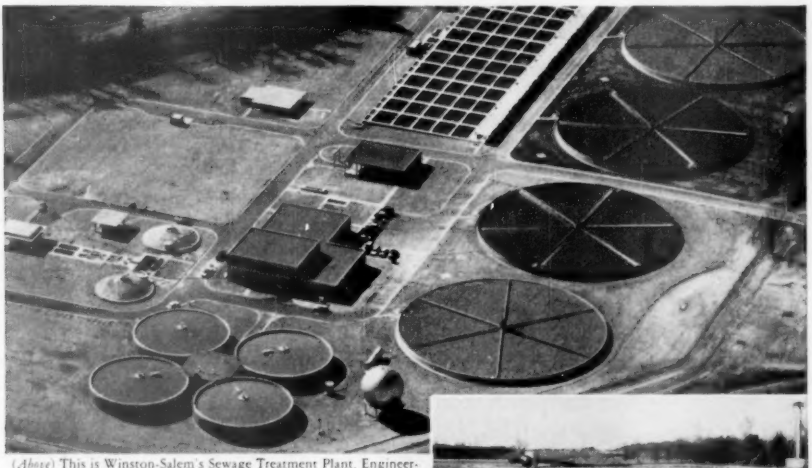

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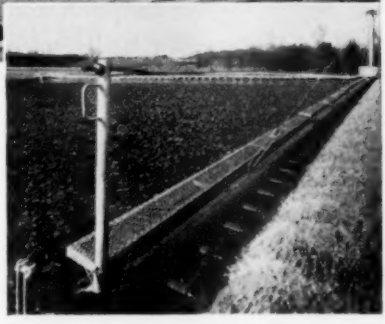

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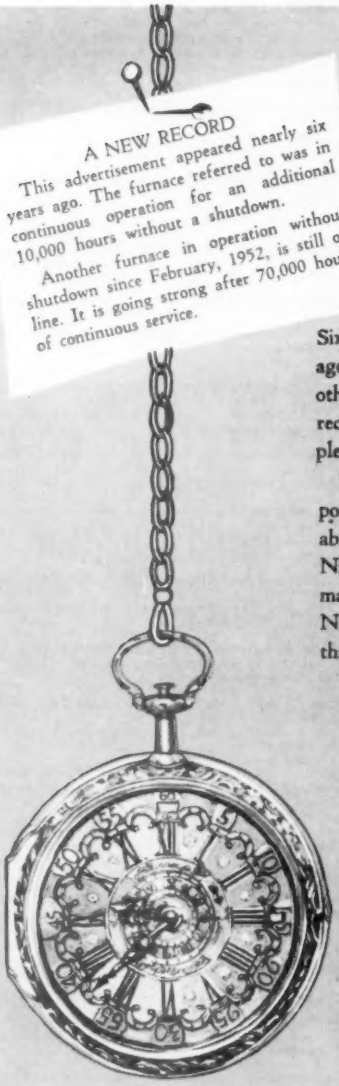

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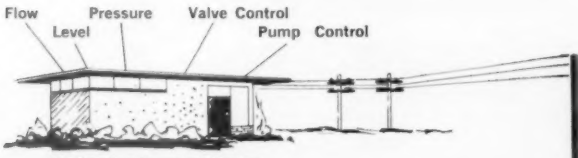

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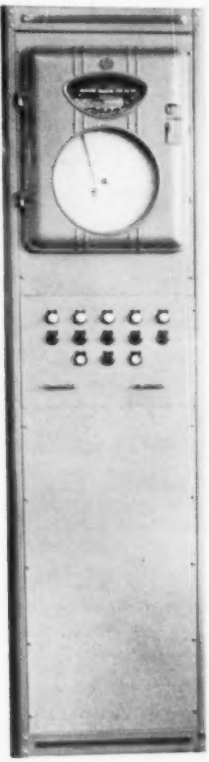

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Association *	*Date*	*Place*
Kansas	Apr. 22–24	Broadview Hotel, Emporia
California	Apr. 27–30	Hotel Claremont, Berkeley
Arizona	Apr. 28–30	Pioneer Hotel, Tucson
Montana	Apr. 28–30	Northern Hotel, Billings
Michigan	May 23–25	Park Place Hotel, Traverse City
Sweden	May 27–29	Inst. för Arbetshygien, Helsinki, Finland
Alabama	June 6–8	60 Auburn Univ., Auburn
Maryland-Delaware	June 8–10	Alexander Hotel, Hagerstown, Md.
Virginia	June 15–16	John Marshall Hotel, Richmond
Central States	June 15–17	Lorraine Hotel, Madison, Wis.
Ohio	June 15–17	Neil House, Columbus
Iowa	June 15–17	Ottumwa Hotel, Ottumwa
ISP (England)	June 21–24	Scarborough, England
Pennsylvania	Aug. 10–12	Pa. State Univ., University Park
South Dakota	Sept. 7–9	Grand Hotel, Watertown
Kentucky-Tennessee	Sept. 12–14	Andrew Jackson Hotel, Knoxville, Tenn.
Missouri	Sept. 25–27	Hotel Governor, Jefferson City
Pacific Northwest	Oct. 20–22	Marcus Whitman Hotel, Walla Walla, Wash.
Rocky Mountain	Oct. 24–26	Broadmoor Hotel, Colorado Springs, Colo.
Canada	Oct. 24–27	Statler Hotel, Buffalo, N. Y.
New England	Oct. 26–27	Taft Hotel, New Haven, Conn.

* See preceding left-hand page for full name.

THIRTY-THIRD ANNUAL MEETING

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Philadelphia, Pennsylvania

Technical Meetings and Exhibits—Convention Hall

October 2–6, 1960

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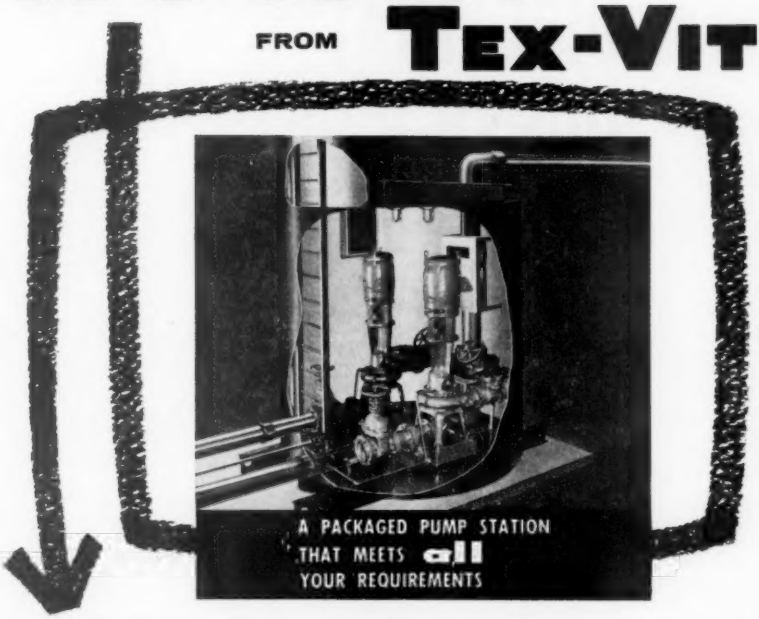


- **ADVANCED DESIGN FOR GREATER EFFICIENCY**
  Compare motor and pump assembly of this duplex sewage pump station with any other and see the TEX-VIT superiority. Individual shaft for each pump and each motor. Four bearings per assembly instead of the customary two. Heavy steel shell, completely protected from corrosion.
- **DIRECT PRESSURE CONTROLS FOR EXTRA RELIABILITY**
  Exclusive TEX-VIT controls are actuated by pressure variations within the wet well. No compressor to maintain; no floats or other moving parts in the wet well.
- **IMPROVED AIR CIRCULATION AND CONDITIONING**
  Extra large blower provides complete change of air every 60 seconds. Air inlet and outlet are at opposite sides of station to prevent short circuiting of air. Refrigeration-type dehumidifier.
- **SUPERIOR COMPONENTS FOR EXTRA DEPENDABILITY**
  Pumps, motors, controls, and other components bear the brand names you would expect to find in a *Who's Who of American Industry*. You know you are getting the best when you specify TEX-VIT.
- **INSTALLATION TESTED AND PROVED**
  TEX-VIT packaged pump stations are setting new standards. Names of muncipal and industrial users furnished on request.
  Let TEX-VIT help you with your sewage lift problems. TEX-VIT also makes pneumatic ejectors and packaged ejector stations.

Write for bulletin No. PS-60

TEX-VIT
SUPPLY COMPANY
MANUFACTURING DIVISION
Dept. S-4, Box 117, Mineral Wells, Texas

PIPE THAT DOESN'T GET COLD FEET IN WET TRENCHES

This cast iron pipe was installed to carry water to an important industrial site in Indiana.

At the time of installation, weather conditions were severe; the trenches were wet and muddy.

Still, the cast iron pipe was easily and speedily assembled, enabling the contractor to complete the job well ahead of the deadline.

Cast iron pipe does not get cold feet over rugged soil conditions. Its rugged design, corrosion-resistant qualities, and bottle-tight joints will withstand the most severe pressures.

Modern, cement-lined cast iron pipe will deliver a continued full flow of water for at least a century . . . and rarely requires repairs or replacement.

CAST IRON PIPE RESEARCH ASSOCIATION
Thos. F. Wolfe, Managing Director
3440 Prudential Plaza, Chicago 1, Illinois

CAST IRON PIPE
THE MARK OF THE 100-YEAR PIPE

Another Eimco-Process design innovation . . .

An annular sludge trough is a key feature of the Type CSPA Oxidator. By positioning the trough outside the aeration zone, both the sludge raking distance and the sludge holding time are shortened and deep, uniform aeration is possible without sludge dilution.

Want good treatment on both low and high flows?

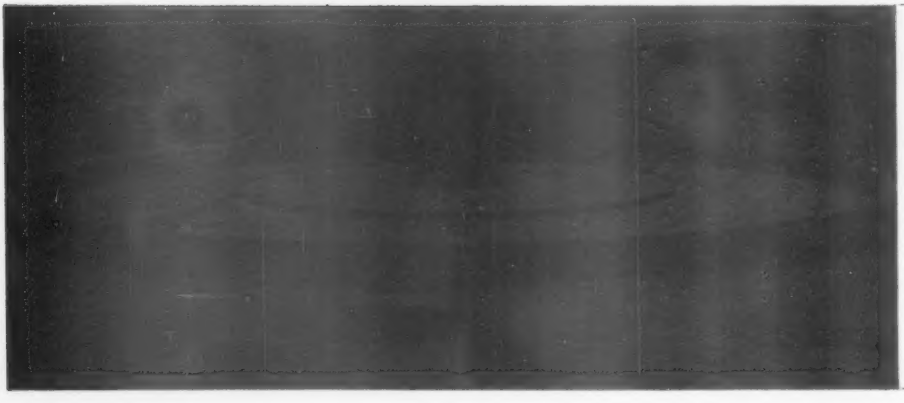
THE OXIDATOR®

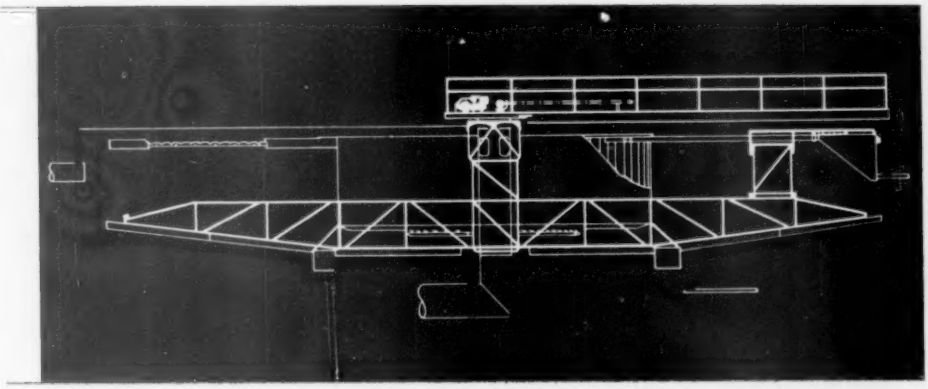
WAS DESIGNED

FOR YOU!

If you are concerned with widely varying flows, or if you are designing for ultimate high capacity yet need good treatment at present low flows, the Oxidator is meant for you. This economical intermediate treatment unit combines aeration, flocculation and sedimentation in a single tank to give high removals at near the cost of primary treatment.

Thorough aeration minimizes scum formation and keeps the influent fresh. Light organics are





quickly flocculated and efficiently removed. And . . . the exclusive annular sludge trough design permits obtainment of a concentrated sludge in a short detention period.

Whether for consideration on an immediate problem or for future reference, you should know about the improved Oxidator. Detailed information is available from any of our nationwide representatives or you may wish to write directly to us in San Mateo.

B-574

. . . another outstanding installation

90 ft. dia. Eimco- Process Oxidator with annular sludge trough in operation at Ponca City, Oklahoma. Consulting Engineers— Pate Engineering Company, Tulsa, Oklahoma.

THE EIMCO CORPORATION

Process Engineers Division

420 Peninsular Avenue
San Mateo, California

FLUIDICS* AT WORK

Checking clarified effluent of the Permutit Colloidair Separator at the Homasote Company

How to side-step a public health hazard . . . and step into big new savings

Maybe you have a problem similar to the one that confronted the Homasote Company, Trenton, N. J.

They make high-density wood fiber insulation board. The process *could* create a waste disposal problem and impair the company's good community relations.

The answer found by Homasote turned out to be more than just a waste treatment system. It is also a valuable way to save money, through reclaimed product and water.

The Permutit Colloidair Separator is behind this accomplishment.

Saves product. White water feeds into the Colloidair, where a "bubbling-out" process raises the fibers to the surface. These fibers are scraped off and reclaimed.

Saves water. No water is wasted in this closed-circuit operation. Colloidair clarifies it for Homasote's re-use in pump glands, stuffing box seals, and for general cleanup.

Saves heat. The reclaimed water holds most of its heat during the clarifying process. This saves the expense of reheating for re-use.

Easy to use. Colloidair works almost by itself. The operator does little more than periodically charge chemical tanks.

Homasote is getting these benefits today . . . when would YOU like to start?

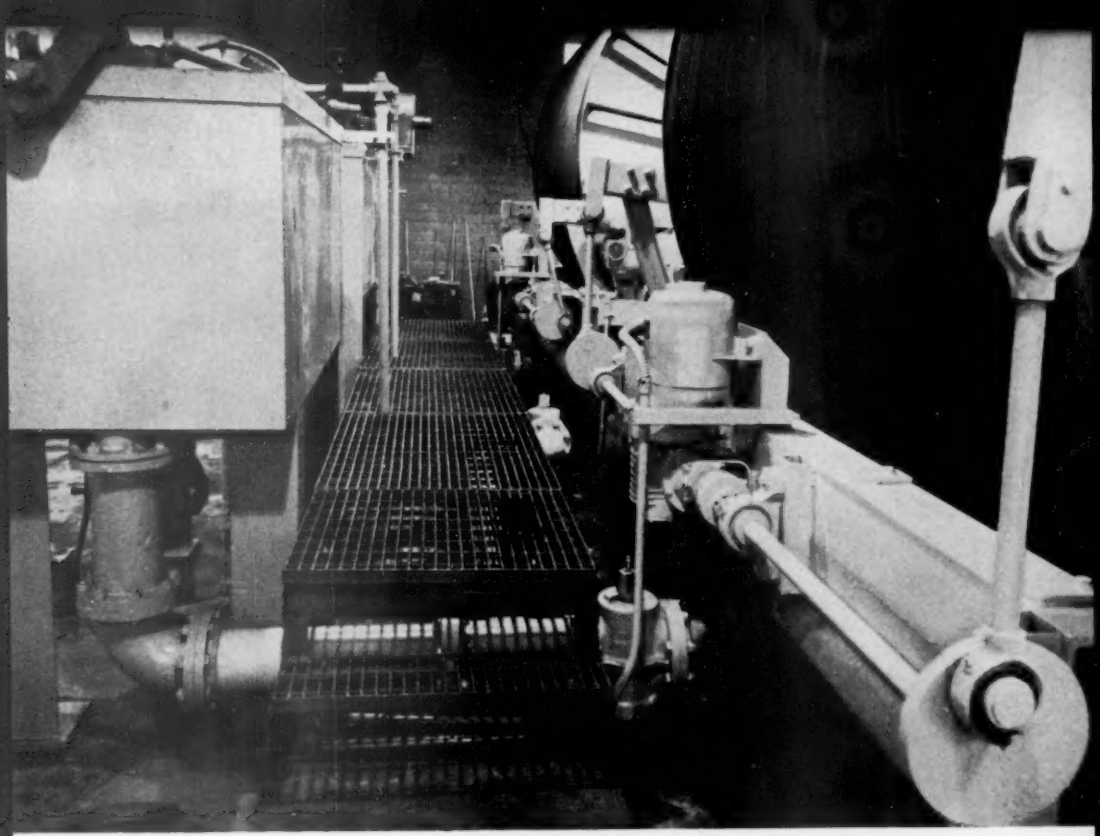
We will be glad to run lab and/or pilot plant tests to show just what the Permutit Colloidair Separator can do with your white waters and/or waste waters.

Save your public relations . . . and your money. Get more information on Colloidair by writing our Permutit Division, Dept. JWP-40, 50 West 44th Street, New York 36, New York.

*FLUIDICS is a new Pfaudler Permutit program that provides a modern, imaginative approach for handling and processing liquids and gases more profitably.

PFAUDLER PERMUTIT INC.

Specialists in FLUIDICS . . . the science of fluid processes

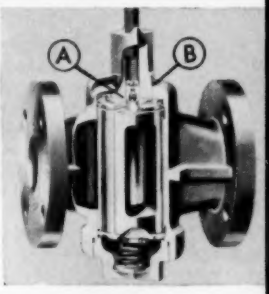


Homestead Lubricated Plug valves at work in Bloom Township Sewage Treatment Plant. Consulting Engineer for installation: Consoer, Townsend and Associates, Chicago.

"Five years—and not a cent for replacement parts!" . . the facts from Sanitary District of Bloom Township

Installed in 1955, Homestead® Lubricated Plug Valves have carried 60,000 gallons of digested sludge from the conditioning tanks to the vacuum filters *every week*. The only maintenance required has been repainting one time.

Homestead's Lubricated Plug Valves mean lower maintenance costs . . . as proved at the Sanitary District of Bloom Township Sewage Treatment Plant in Chicago Heights, Illinois. Homestead Valves can mean lower maintenance costs of *your* pipe lines, too. Write today for complete information.



Cross-section of Homestead Lubricated Plug Valve. Triple head seal (A) prevents leakage. Tell-tale lubricant ring at (B) signals full system, prevents waste of lubricant and contamination of line fluids.

hv

HOMESTEAD VALVE MANUFACTURING COMPANY
P. O. Box 48 • Coraopolis, Pennsylvania

Please send me catalog on Homestead Lubricated Plug Valves. We are interested in this specific application:.....

.....

Name.....

Street.....

City.....Zone.....State.....

48

K&M ASBESTOS-CEMENT SEWER PIPE

PERMANENTLY

Sewer pipe maintenance headaches disappear when you install "K&M" Asbestos-Cement Sewer Pipe.

Flow-stopping, pipe-clogging roots . . . no matter how cobwebby-thin . . . can't penetrate the exclusive patented "K&M" FLUID-TITE coupling . . . unmatched in its tight sealing. In repeated tests, "K&M" Asbestos-Cement Sewer Pipe successfully resisted infiltration even when external water pressure was 25 psi . . . the equivalent of a 58-foot head and way above field conditions.

Service remains continuous and uninterrupted . . . with fewer inspections and periodic cleanings. You can reduce treatment loads and costs . . . because infiltration is eliminated. Being made of quality asbestos fibers and portland cement, "K&M" Sewer Pipe does not corrode and is immune to electrolysis.

In fact, "K&M" Asbestos-Cement Sewer Pipe is a tax-saver at every stage of construction: planning and installation, as well as maintenance. Transportation and handling is less expensive, because "K&M" Asbestos-Cement Sewer Pipe is light in weight.

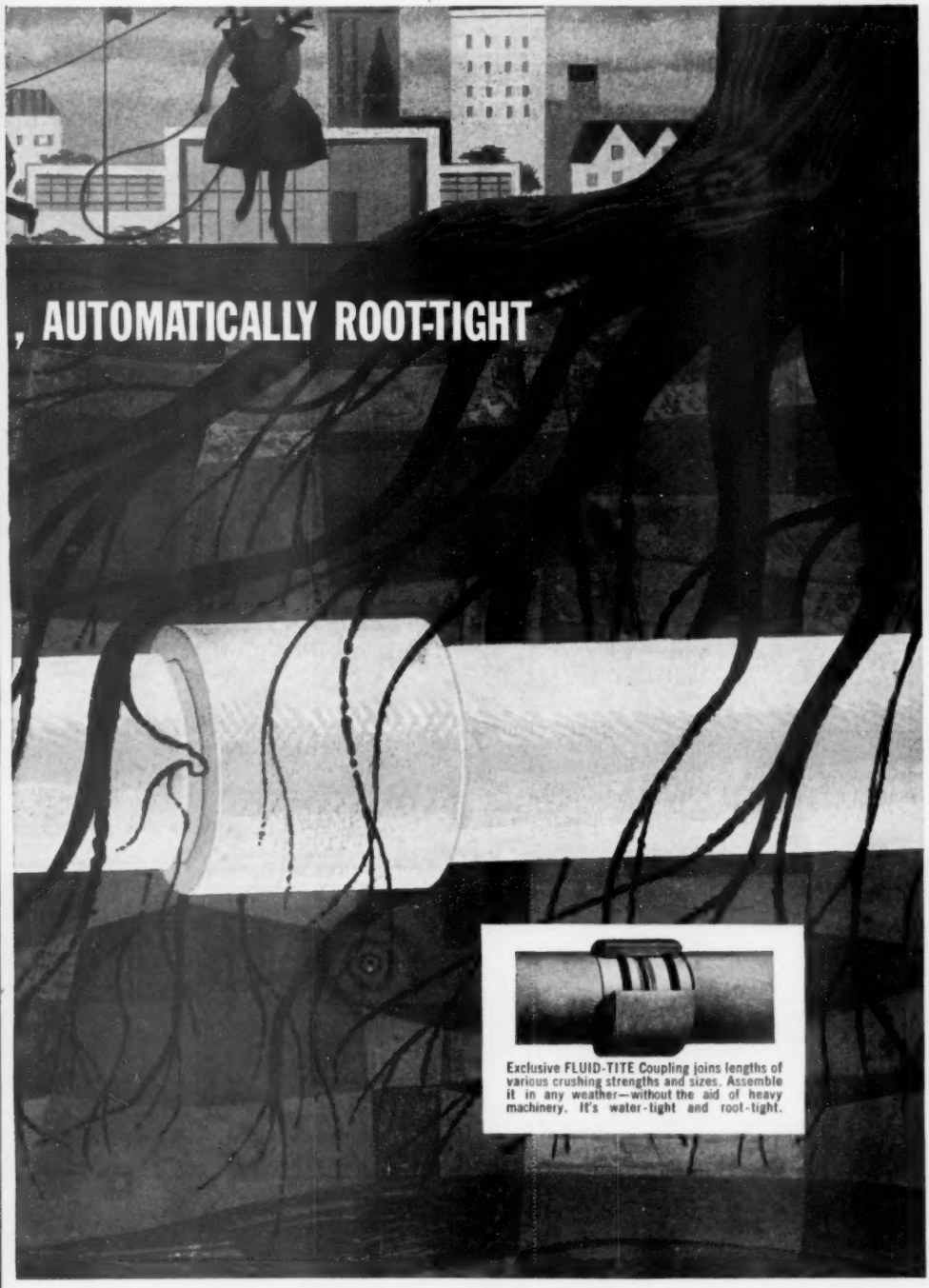
In short, practically indestructible materials plus eighty-five years of asbestos engineering give you the finest, most dependable sewer pipe available. That's why we urge you to write today for more information. **Dept. P-1740**

5 CRUSHING STRENGTHS OF K & M SEWER PIPE

Size Inches	Class 1500	Class 2400	Class 3300	Class 4000	Class 5000
6	1500	2400	3300		
8	1500	2400	3300		
10	1500	2400	3300	4000	5000
12	1500	2400	3300	4000	5000
14	1500	2400	3300	4000	5000
16	1500	2400	3300	4000	5000

K·M BEST IN ASBESTOS

KEASBEY & MATTISON
Company • Ambler • Pennsylvania


, AUTOMATICALLY ROOT-TIGHT
Exclusive FLUID-TITE Coupling joins lengths of various crushing strengths and sizes. Assemble it in any weather—without the aid of heavy machinery. It's water-tight and root-tight.

Springfield, Illinois, sewage plant. In foreground, three Verti-Flo Clarifier Basins. *Inset:* Partitioning baffles divide tank into efficient vertical-flow cells. V-notched weirs control cell-to-cell flow, minimize short-circuiting. Each basin equipped with two longitudinal and one cross collector.

C. C. Larson, district manager, Springfield Sanitary District; A. Paul Troemper, district engineer; Jenkins, Merchant & Nankivil, consulting engineers.

REX® Verti-Flo® Clarifiers boost capacity, reduce space

Thousands of dollars in valuable plant space were saved when consulting engineers and the management of the Springfield, Illinois, Sanitary District selected Rex Verti-Flo Clarifiers for their recent plant expansion. In addition, their selection provides for economical plant growth in the future.

Rex Verti-Flo's unique design gives Springfield over twice the capacity of conventional clarifiers—in the SAME amount of space.

An exclusive operating principle provides this increased capacity: a system of multiple cells changes the flow from horizontal to vertical. The main body of each Verti-Flo tank is proportioned into four cells with a generous weir length around each.

Another important reason why Verti-Flo operates efficiently at higher overflow rates is this: it allows a tank to be hydraulically balanced after construction.

Easily Adapted to Existing Tanks

Existing tanks can be readily converted to the Rex Verti-Flo design. Better results are assured and increased capacity is guaranteed. Both cost and alterations to existing structures are minimized.

Send for Full Information

To obtain maximum capacity in any old or new plant, find out about Rex Verti-Flo Clarifiers. Write CHAIN Belt Company, 4606 W. Greenfield Avenue, Milwaukee 1, Wisconsin.

REX®

CHAIN BELT COMPANY

IF YOU DON'T HAVE A TRUCK-LODER
Your Sewer Crew is WORKING TOO HARD!

PATENT 2,892,557

LOADS DIRECT INTO TRUCK

ENDS DUMPING SEWERAGE ON STREETS!

RELIEVES WORKERS OF 3 HARD JOBS!

The **TRUCK-LODER** frees your workers from the hard jobs of manually dumping the bucket on the street — shoveling deposits into the truck, and cleaning up the street.

With the **TRUCK-LODER** two men can accomplish far more than four with conventional bucket machines, thereby freeing the extra men for other important sewer maintenance work.

Users report the elimination of time-consuming operations has increased sewer-cleaning capacity by 40%. They agree that **TRUCK-LODER Bucket Machines exceed all performance estimates.**

Your Flexible representative will be glad to give you facts and figures at your convenience.

Use With Your Present Machines

Use Truck-Loders on the dumping end and your regular machines on the Pull-In end. Net result—two complete, modern Truck-Loder units.

FLEXIBLE INC.

3786 DURANGO AVE., LOS ANGELES 34, CALIF.
1005 SPENCERVILLE ROAD, LIMA, OHIO
415 SOUTH ZANGS BLVD., DALLAS, TEXAS

"They wuz a little slow beginnin'... but look at 'em lay Tyton now!"

U.S. cast iron **PIPE** FOR WATER, SEWERAGE AND

everybody's doin' it

Laying Tyton Joint® pipe, that is. This easy-to-assemble pipe is so foolproof even a bear can get in the act. Only one accessory needed—a simple rubber gasket. No bell holes. No caulking equipment. No nuts or bolts to fiddle with. No weather worries, either. Tyton doesn't mind wet feet . . . can be laid in rain or wet trench. Result: more working days, more lengths laid per man hour, lower installation cost. Sound interesting? It is! Call or write for the facts.

U.S. PIPE AND FOUNDRY COMPANY
General Office: Birmingham 2, Alabama

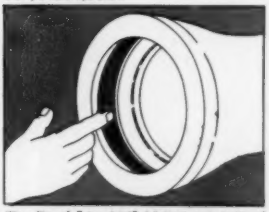
A Wholly Integrated Producer from Mines and Blast Furnaces to Finished Pipe.

INDUSTRIAL SERVICE CAST IRON®

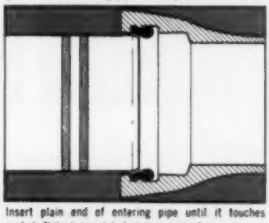
TYTON®

ONLY FOUR SIMPLE ACTIONS

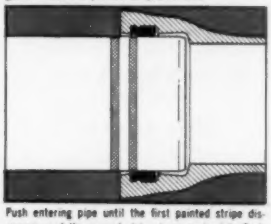
Insert gasket with groove over bead in gasket seat . . . a simple hand operation.



Wipe film of Tyton Joint® lubricant over inside of gasket. Your receiving pipe is ready.

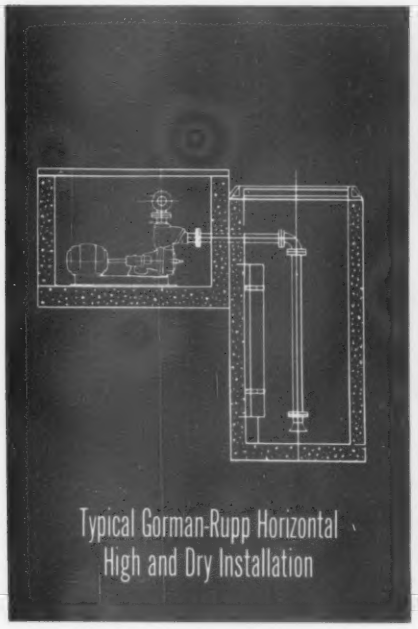


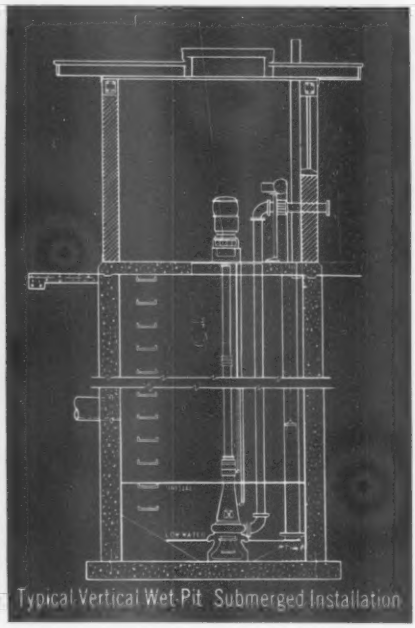
Insert plain end of entering pipe until it touches gasket. Note two painted stripes on end.



Push entering pipe until the first painted stripe disappears and the second stripe is approximately flush with bell face. The joint is sealed . . bottle-tight, permanently! The job's done . . . . fast, efficiently, economically. Could anything be simpler?

SAVE UP TO 50% . . .
GORMAN-RUPP SEWAGE PUMPS


Typical Gorman-Rupp Horizontal High and Dry Installation


Typical Vertical Wet-Pit Submerged Installation

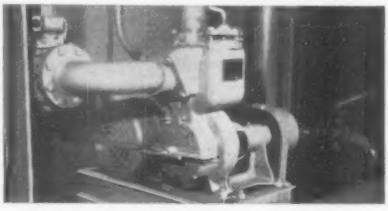
A glance at the two diagrams above will quickly show simplicity of installation. Further study of equipment costs will add to the proof of savings when you install Gorman-Rupp High-and-Dry Sewage Pumps. These economies are possible through design features which permit self-priming in lifts up to 15 feet, positive action, and nearly complete avoidance of clogging.

Gorman-Rupp Sewage Pumps need almost no maintenance compared to submerged pumps. No need to service by entering pit or raising pump. Just a turn of the handle and the end plate is off.

In lift stations and sewage plants, these Gorman-Rupp units have shown amazing savings . . . operating where others have failed. 2″, 3″, 4″ and 6″ pumps—connected to your power or ordered complete with power units. Write for specifications.

THE GORMAN-RUPP COMPANY
305 Bowman Street • Mansfield, Ohio
Gorman-Rupp of Canada, Ltd., St. Thomas, Ontario

In Jacksonville, Florida, a 3-inch Gorman-Rupp pump replaced a submerged pump after 2½ years in service.



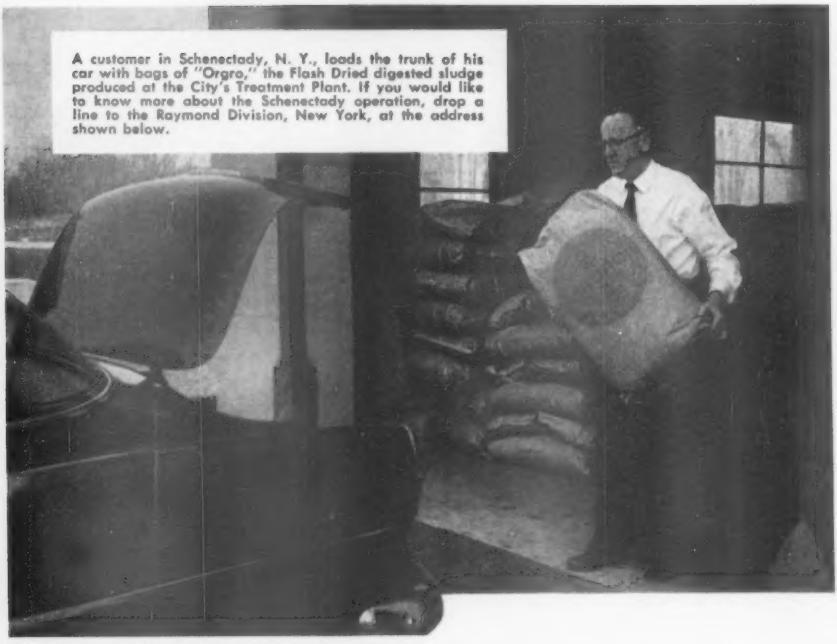
ELECTRICALLY CONTROLLED CHLORINATION SYSTEMS

The commands are electric. The operation is automatic. Engineered by F & P . . . chlorination goes modern.

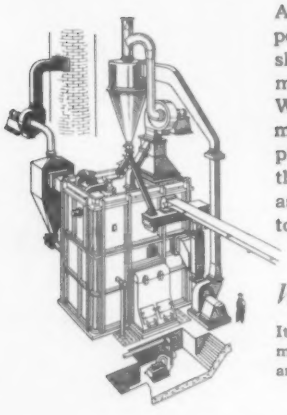
Regulate chlorine feed rate *precisely, automatically* with electrical signals from program controllers, multiple and step rate controllers, automatic proportioning controllers, remote controllers, magnetic flowmeters, and residual analyzers. Don't invest in *any* chlorination system until you check out F & P Electrically Controlled Chlorination Systems. Fischer & Porter Company, 1140 Fischer Road, Hatboro, Pennsylvania.

FP FISCHER & PORTER COMPANY *INSTRUMENTATION AND CHLORINATION*

These MANUFACTURING AFFILIATES carry Fischer & Porter engineering designs to the far corners of the earth: FISCHER & PORTER LTD., WORKINGTON, CUMBERLAND, ENGLAND • FISCHER & PORTER G.m.b.H., GROSS-ELLERSHAUSEN, BEI GOTTINGEN, GERMANY • FISCHER & PORTER N.V., KOMEETWEG 11, THE HAGUE, NETHERLANDS • FISCHER & PORTER Pty. LTD., 184 HANNA ST., SOUTH MELBOURNE, VICTORIA, AUSTRALIA.


A customer in Schenectady, N. Y., loads the trunk of his car with bags of "Orgro," the Flash Dried digested sludge produced at the City's Treatment Plant. If you would like to know more about the Schenectady operation, drop a line to the Raymond Division, New York, at the address shown below.

Can you think of a better way to dispose of sewage sludge?



At first glance, the trunk of a car may seem to be a peculiar place to deposit sewage sludge. But when the sludge is dry, sterile and neatly packaged, it can be a marketable commodity rather than a bothersome waste. When uncontaminated with noxious industrial materials, it is an excellent soil conditioner. Its sale produces revenue for the treatment plant, eliminates the nuisances of drying beds, lagoons and ash disposal and, at the same time, renders a service to the people of the community.

What's the best method of drying sludge?

It's the C-E Raymond Flash Drying System, a system used by communities large and small across the nation, a system proved in service and one which dries more filter cake than all other systems combined.

COMBUSTION ENGINEERING

RAYMOND DIVISION, 1132 West Blackhawk St., Chicago 22, Illinois
Eastern Office: 200 Madison Avenue, New York 16, N. Y.
Western Office: 510 West Sixth Street, Los Angeles 14, Cal.
Canada: Combustion Engineering-Superheater Ltd.

C-159A

ALSO FLASH DRYING AND INCINERATION SYSTEMS FOR INDUSTRIAL WASTE DISPOSAL

Specify
IOWA
for every job
Iowa's complete line of valves for treatment plants assures you of efficient operation, low maintenance cost, and replacement parts when needed in years to come.

IOWA CHECK VALVES—Swing gate type for use in either horizontal or vertical pipe lines. Bronze trimmed throughout. Stainless steel hinge pins. Leather or rubber faced gates, or solid bronze, as desired—aluminum gates available for air service.

IOWA MEDIUM AND LOW PRESSURE VALVES—either non-rising stem or outside screw and yoke types. Designed as carefully as those conforming to A.W.W.A. specifications but with lighter construction for lower pressures.

Write TODAY for descriptive literature

IOWA FLAP VALVES—all iron; can be furnished with bronze hinge bolts or fully bronze mounted—including bronze bolts, flap ring and seat ring—when required.

CLOW HYDROSTATIC RELIEF VALVES — commonly used in bottom of concrete tanks to prevent ground water from floating the tank. Close-grained cast iron body and lid; lead seat ring and lid ring. Cast iron grate in bottom prevents foreign matter from entering tank.

IOWA PLUG DRAIN VALVES—either rising or non-rising stem type. High strength cast iron bodies; bronze stem, operating nut, disc ring and seat ring; rustproof steel bolts and nuts. Handwheels, extension stems, plain or indicating floor stands if required.

IOWA SLUICE GATES—can be used in lines having both seating and unseating pressures. High strength cast iron, bronze mounted, with solid bronze adjustable wedges. Can be equipped with cylinder or motor unit for automatic operation.

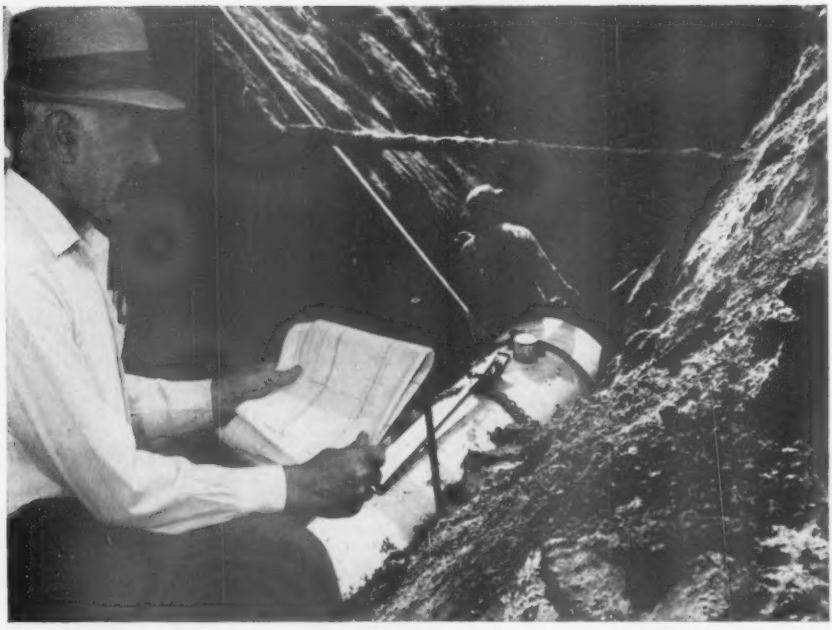
IOWA SHEAR GATES—either all iron or iron, bronze mounted. Gate-seating wedges bolted on, permitting replacement without replacing frame.

For 50 years a reliable source for precision engineered products

IOWA VALVE COMPANY

A Subsidiary of James B. Clow & Sons

Oskaloosa, Iowa



"Not a 'comeback' in 18 years" ...with Transite Sewer Pipe

says Anthony Garcia, President
Allerton Construction Co., New York

"In my eighteen years of installing Transite sewer mains and house laterals, I have never had a 'comeback' because of line failures. Most of the installations have been 100% under water. Therefore, tight joints that stay tight are absolutely essential! For these reasons Transite has always been my choice for sewer line service. I never worry about passing leakage or infiltration tests. I play it safe—with Transite."

You'll want to read how Johns-Manville Transite® Pipe has helped other contractors increase their profits and productivity . . . in the informative and interesting new, 8-page booklet, TR-206A. Send for it now! Address Johns-Manville, Box 14, New York 16, New York. In Canada, Port Credit, Ontario.

JOHNS-MANVILLE JOHNS MANVILLE JM PRODUCTS

CUTS SEWAGE TREATMENT COSTS UP TO 27%

Gilbert W. Clifford
Gilbert W. Clifford & Associates
Treasure Island, Florida
Designed around Hi-Cone® for the Carol City, Florida plant

F. M. Cahaly
Fay, Spofford & Thorndike
Boston, Massachusetts
Chose a Cavitator® plant for Kent County Hospital, Warwick, R. I.

Marion C. Welch,
Cons. Engr.
Lexington, Ky.
Specified Hi-Cone® for the Lansdowne Subdivision, Lexington, Kentucky

Richard A. Stevens, A.I.A.
Stevens & Bertin, Architects
Rochester, New York
Kingdom Plaza Shopping Center, Seneca Falls, N. Y.

Harold D. Briley (left) and Harry E. Wild (right)
Briley, Wild & Assoc.
Daytona Beach, Florida
Specified Hi-Cone® for Clearwater's Marina plant

- Lowest capital costs
- Shortest aeration period; lower power demand
- Tank walls and floor continuously scoured
- Lowest operating and upkeep costs
- No falling off in efficiency
- No diffusers, blowers, or air cleaners
- No diffuser cleaning
- No expensive blower housing

Call in your Yeomans representative to fill out details on the cost story, or request the performance reports offered at right.

YEOMANS

☐ Cavitator® Engineering Report: technical analysis and report covering cost and performance.

☐ The Hi-Cone® Studies: Dr. W. E. Howland's Study and Commentary on the Manchester Reports.

YEOMANS Brothers Company
1999-7 N. Ruby St., Melrose Park, Ill.

Please send me copies of the reports checked above:

Name_____

Title_____

Organization_____

Address_____

City_____ Zone_____ State_____

T-5923

How Lincoln, Ill., sewage plant **SOLVED**

HEAVY GREASE LOAD PROBLEM

with one Dorrco Vacuator®

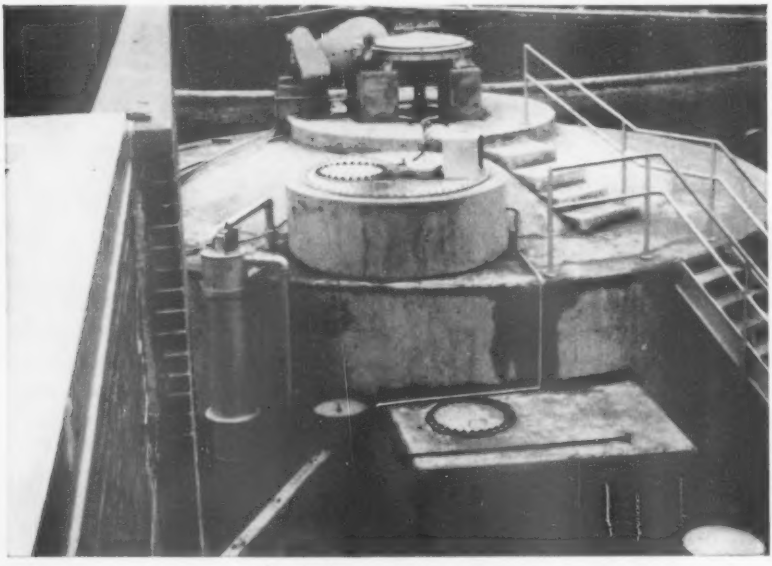
A large institution's decision to grind all its garbage threw a monkey wrench into Lincoln's plans for improving its sewage treatment plant. The institution was already a heavy grease contributor. Increased strength of the sewage would be tremendous. Calculations showed that Lincoln must be prepared to handle wastes equal to those of a town double its size!

Could the town alter its plans, handle the heavier loads and do it efficiently and economically?

It did – with the help of Dorr-Oliver equipment. In one major move a 26′ dia. Dorrco Vacuator was installed for primary treatment and the original settling tanks became intermediate tanks. In the first year of operation, BOD removal through the Vacuator ranged from 19 to 27% – averaging 22%. Suspended solids removed averaged 34%. Not only that, the Dorrco Vacuator proved it could reduce digester requirements by pumping thick sludge averaging 3.6% in solids content to the digester. In short, the Dorrco Vacuator was a vital factor in solving Lincoln's problems – for years to come.

Complete information on the Dorrco Vacuator is readily available. Ask for Bulletin No. 6301. Or write for details on the complete line of Dorr-Oliver equipment for sewage treatment.

Plant Superintendent: Lawrence Arthur. Consulting Engineers: Crawford, Murphy & Tilly, Springfield, Illinois.



DORR-OLIVER

WORLD-WIDE RESEARCH • ENGINEERING • EQUIPMENT

Sewage Works

SANITATION IN SPACE *

By Laurel van der Wal

Space Technology Laboratories, Inc., Los Angeles, Calif.

Man will soon leave Earth's atmosphere and travel in space. In the future, colonies will be established on other planets, and perhaps eventually in other galaxies. But, however far the space traveler goes, he will always be subject to the physical and psychological limitations of man. It is possible that given enough time, these limitations may be eased through genetic adjustment of man himself to a particular set of environmental conditions. An example of this is the characteristic "barrel-chest" of Peruvian Indians who live in the Andes—an adaptation to the low oxygen content of air at 14,000-ft altitude. However, in space far more than on Earth, man will encounter a wide variety of environments which are not in their natural state conducive to his survival. Man must use his skill, experience, and ingenuity to transform the conditions of space and of other worlds into environments where it is possible not only to survive but to exist comfortably.

It is essential to man's health on Earth that he maintain sufficient control over the disposal of waste, both biological and industrial, to prevent disease. As long as man was nomadic and there were no large population centers, the problem was not critical. It was necessary only that man not contaminate his sources of food and water, and the likelihood of this was small since he was continually changing his dwelling place. As man populated and overpopulated the choicer dwelling sites, the occurrence of disease of epidemic proportions as a consequence of improper waste disposal increased. Even today, epidemics continue to ravage areas of the world where the problems of sanitation have not been adequately met. However, in the United States, as in numerous other countries, standards of sanitation have been developed which successfully maintain a healthy environment for man on Earth. Here then is the skill and experience necessary to resolve the problems of providing a healthy environment for man in space.

The Basic Problem

There is, however, one important distinction between these two problems of sanitation on Earth and in space. On Earth, it is only necessary to prevent contamination of an environment which is basically a compatible one. In the regions of space and on other worlds, the natural environment will be hostile to man. Therefore, it is necessary first to provide a friendly environment and then, as on Earth, to protect this environment against contamination.

How urgent is this problem of developing such an environment-control system for manned space flight? From the biomedical standpoint it is possible today to keep a man alive in a small satellite capsule for several

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hours, perhaps for as long as two days. Unfortunately, the reliability of the vehicles which must be used to launch a manned capsule into orbit is not high enough to warrant an attempt today. But the vehicles have been constructed and are being tested; no conceptual problems remain to be solved. And the current flight-test programs are specifically aimed at improving reliability. It will not be long before manned space flight, on a short term basis, will be a reality.

Performance calculations show that these vehicles now being tested can also be used to send an appreciable payload to Mars. But a manned mission to Mars would last about three years; 260 days for the trip from Earth, 464 days on Mars awaiting proper positioning of the two planets for launching the vehicle from Mars, and 260 days for the return voyage. There is no environment-control system now available which can maintain a closed ecology for so long a time. Furthermore, there are a number of conceptual problems in the operation of such a system which are yet unresolved. This, then, is a major obstruction to successful achievement of extended space flight in manned vehicles.

Vehicles for Space Travel

Figure 1 is a qualitative representation of current vehicle capabilities, as an indication of the nature of the problem.

The solid lines represent the payload capability of three types of currently available ballistic missiles, plus added stages, when used for the missions indicated on the abscissa. The crosshatched bands represent estimates of payload requirements for maintaining one man and two men on the respective missions. For example, a velocity of about 26,500 fps is required to place a satellite in orbit at 200 nautical miles altitude. This velocity can be achieved by an Intermediate Range Ballistic Missile, plus added stages, carrying a payload which is sufficient to sustain and recover one

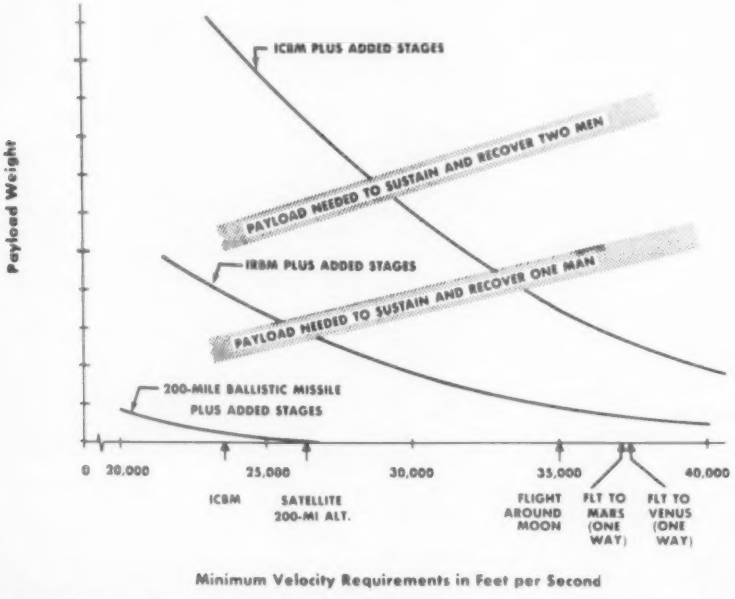

FIGURE 1.—Effect of velocity requirements on payload-carrying capacity.

man. If a flight around the moon is considered (requiring a velocity of 35,000 fps), it is seen that an Intercontinental Ballistic Missile, with added stages, can carry an appreciable payload on this mission, but the payload is not great enough to permit a man to be included in the vehicle. In order to extend the man-carrying capability of space vehicles to more sophisticated space missions, two possibilities exist. Either the load-carrying capability of the vehicles must be improved, which would move the solid lines upward on the graph, or—of more interest here—the weight requirements for sustaining the man must be reduced, thus moving the bands downward. Either or both of these improvements would permit attainment of higher velocities with the required payloads, and thus make possible more ambitious space missions than can now be foreseen.

Environment-Control Problems

Man's initial venture into space will be in a tiny space capsule, at first for only a few hours, but later for periods of days, weeks, or more. The next step will be a permanent space station, and visits to and colonization of other planet worlds will follow. The problem of sanitation in space may be approached by considering the characteristics of each of these environments, their basic differences from the normal Earth environment, and the space applicability of current solutions for sanitation problems on Earth.

The most difficult environment-control problems are those which must be solved for the space capsule, rather than for the space station or moon colony; as the size of the controlled environment increases, the weight and volume allowed for the solution of these problems will increase, permitting refinements to produce a situation more closely approximating that on Earth.

First, picture the intrepid space-man confined in a tiny capsule no larger than a telephone booth, traveling at velocities up to 35,000 fps through field-free space, along a trajectory which passes through widely differing regimes of temperature and pressure. The capsule is for this man a terrella, "a little Earth" which must provide all that he needs to remain alive and well. This little Earth has no oxygen-rich atmosphere and no gravitational field. It will be contaminated by gaseous, liquid, and solid wastes produced by man's normal metabolic processes, by preparation of and residue from food, by washing and cleaning, by operation of component equipment, and so on. It will also be exposed to contamination by cosmic radiation from space and, if nuclear rockets are used, from the propulsion system or auxiliary power supply. It is an Earth whose weight and volume are strictly limited by the payload capabilities of the launching system for any given mission.

In one sense, the complex of problems in the capsule may be viewed as a miniature or model of the sanitation problems that are beginning to be evident in large cities today. The quantity of waste resulting from the growing population and increasing industrialization requires new techniques for its disposal. Waste products on Earth eventually contribute to the sustenance of living organisms through natural cyclic processes occurring over long periods of time. Water shortages and fertilization needs call for the speeding up of this natural process of reclamation by techniques which may contribute to the solution of recycling problems for space applications. Similarly, as the air in the capsule must be treated to prevent excessive concentrations of carbon dioxide, water vapor, and other toxic or unpleasant gases, so too the air pollution in cities must be decreased. The techniques of smog control are probably not directly useful because of the scale of the prob-

lem and the nature of the contaminants. And there are other factors to be considered: (*a*) the net pressure in the capsule will be half, or less, the normal atmospheric pressure on Earth; and (*b*) nitrogen may be replaced by helium as the inert component of air in the capsule, or pure oxygen may be used.

As another example, just as man in the capsule must be protected from cosmic radiation, so too must man on Earth devise means of protection from the increasing radiation level due to radioactive fallout.

Such new problems at the frontiers of sanitation on Earth are in many respects analogous to the difficulties faced in the design of a system for space applications. The margins of operation in cities grow smaller as populations increase. The boundaries of the world are tightening. At the same time, new problems of radiation, reclamation, and air pollution are arising. The basic elements of the Earth-sanitation problem are observed in their most critical form in the capsule problem.

Space and City Sanitation

There are important differences, however, between the problems of sanitation in space and in modern cities. These differences are defined first by the physical environment and second by the state of missile engineering as reflected in the payloads which can be carried on space missions.

The most important physical difference arises from operation of space sanitation systems in gravitational fields significantly different from that at the surface of Earth. For example, the designer of a sanitation system to be installed on the moon would have to keep in mind that everything on the moon weighs one-sixth its normal weight on Earth. This would, of course, affect hydraulic design. On larger planets, such as Jupiter, the gravitational field would be much stronger than Earth's with corresponding problems. The major difficulties from gravity changes will be encountered when the space vehicle is in a zero gravity field, as in all satellite flights (whether small capsules or large permanent space stations), and in the coast phase of flights between planets.

Design Criteria

Weightlessness

Sanitation equipment to be used in space must be designed to operate under conditions of weightlessness. This would mean, for example, that there would be no gravity head to force water out of a tap when the valve is opened. If a positive pressure feed were provided to force water out of a pipe, there soon would be globules of water ricocheting all over the capsule. Also, aeration processes commonly used for treatment of urban waste products would be seriously disrupted under zero gravity conditions. Gaseous oxygen introduced into a waste solution would probably oxidize the organic components as on Earth. However, the products of this combustion would not rise to the surface because the gas and liquid would weigh exactly the same amount per unit volume in a zero gravity field—namely nothing. This is well illustrated by some studies performed by the Air Force at Wright Development Division using the ingenious "parabolic-arc technique" to obtain short periods of weightlessness (1).

Absence of Atmosphere

A second physical characteristic of space which will affect sanitation system design is the total absence of any external atmosphere. Obviously the common procedure of ventilating an area with large volumes of air at high flow rates to remove undesirable gaseous contaminants cannot be used in space. Furthermore, organic material jettisoned in space will not decompose no matter how long it stays there.

Supply Limitations

This suggests the third physical principle which must guide the designer of equipment for space sanitation. Not only is it impossible, in an absolute sense, to dispose of matter in space, it is also impossible, for the present at least, to collect matter from space which can be used in waste processing. There are no external supply warehouses. All necessary materials must be carried in the capsule from the surface of Earth as a part of the strictly limited payload.

A corollary to this condition is that man must live with everything which is included in the capsule when it leaves the surface of Earth. Not only must every precaution be taken to eliminate known potentially harmful bacteria from the system, but the possibility of changes in the nature of normally benign organisms, as a result of space conditions, must be considered. Especially is this true of organisms which reproduce a generation every few hours. Some research is needed to investigate the mutation rates of harmless bacteria when exposed to the cosmic radiation intensities anticipated in space. It is not inconceivable that mild common-cold germs which man harbors on Earth may, after a few days, and numerous generations, in space, be converted to forms which cause previously unknown and fatal maladies.

Weight and Space Limitations

Finally, the equipment must be designed to occupy a minimum space and must be as light as possible. The importance of these volume and weight conditions cannot be overemphasized. As was shown in Figure 1, the space mission capability of current rocket-powered launching vehicles is a direct function of the payload weight.

Two and a half years ago a forerunner of the first space capsule was used to carry a man to the edge of space in the Manhigh II balloon flight (2). This program was also critically limited in payload weight, but certainly no more so than the initial space capsule program. Figure 2, which shows the Manhigh II capsule, clearly

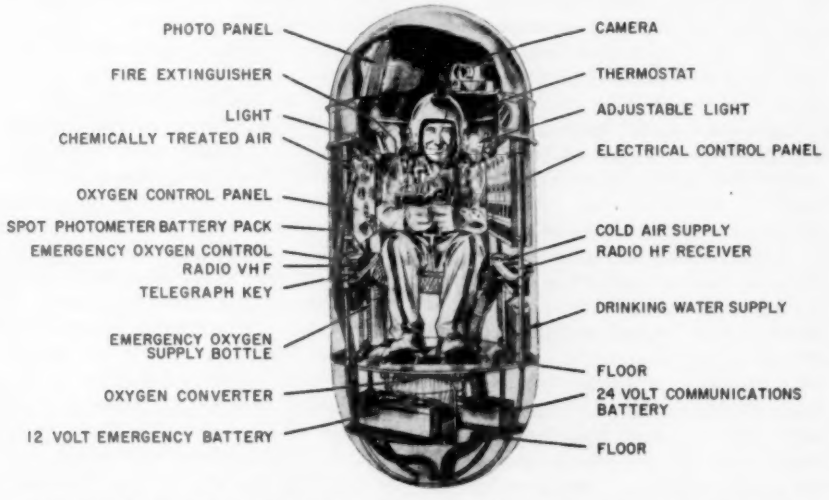

FIGURE 2.—Artist's schematic drawing of the USAF-Winzen Research Inc. MAN–HIGH gondola showing location of internal instruments and components.

indicates that there is very little room for additional equipment.

Possible Solutions to Problems

It is now in order to consider some of the possible solutions to the capsule sanitation problem, keeping in mind the four controlling conditions of weightlessness, absence of atmosphere, minimum volume and weight, and the lack of external supply sources.

Open Cycle System

At present, life support systems are capable of only a few hours' operating duration. Research and development now under way may extend this period to perhaps a week. Current methods of handling wastes, for flights of a few hours, include dietary control of fecal material, storage of urine in ducts built into the subject's suit, and chemical absorption of water vapor and carbon dioxide. No provision is made for personal hygiene or for control of trace poisons which would become critical if allowed to accumulate on longer missions. Food, water, and oxygen are stored separately and supplied as needed, with no reclamation or recycling. The weight penalties imposed by these techniques are a major factor in limiting man's flight duration capability to a maximum of about two days.

This open cycle system, including storage but no reuse of waste products, is the technique used in the Manhigh II flight capsule. The weight breakdown for this capsule is shown in Table I. The indicated weights are those needed to support a man for a period of 48 hr, including an environment-control system weighing about 200 lb plus power.

It has been reported that the payload allowance for the rocket which will carry the first man into orbit is about 2,200 lb, or 600 lb more than the total weight of the Manhigh II capsule. However, the additional weight allowance is not sufficient for the design changes which could convert the balloon gondola to a satellite vehicle. These include additional structure to withstand the more severe flight loads, a re-entry heat shield, a more elaborate recovery system, modification of component equipment for operation in weightlessness, etc. Therefore, the life support system used in the space capsule must be much more efficient than that used on the Manhigh II flight, even for short-term space ventures where open cycle systems are suitable. An absolute maximum weight allowance might be about 80 lb/ man-day, including the required power supply. Even if design improvements produced a system weighing only 50 lb/man-day, it is evident that the open cycle system is not suitable for long duration missions.

TABLE I.—Manhigh II Capsule Weights

Equipment and Supplies	Weight (lb)
Man and personal equipment	250
Structure	200
Instrumentation and communication	300
Power supply	600
Experiments	60
Oxygen supply	70
Air purification system	120
Food and water	20
Total	1,620

Even for flights of only a day or two, storage of waste products becomes a problem. It might appear that the easiest way to dispose of waste would be to jettison it. There are several minor reasons why this would be undesirable. Waste ejection would require the use of an air lock to limit the escape of the inner atmosphere of the capsule into space as the package is being thrown out. But some capsule atmosphere would inevitably be lost and would have to be replaced from storage. Also, it must be remembered that material thrown into space will not decompose, and that it will continue to move at the velocity it has upon its release from the capsule. Con-

sequently, material dropped over the side with no significant velocity component with respect to the vehicle would remain in a fixed position relative to the capsule. Aside from aesthetic considerations, the Martians would probably not be very impressed if the first Earthship arrives in a cloud of refuse. This concept was illustrated a century ago by Jules Verne in his story of a trip to the moon. One of two dogs taken along by the moon voyagers died en route and was "put outside." The dead dog stayed there for the remainder of the trip—a rather gruesome reminder that one cannot use Earth techniques for waste disposal in space.

However, the major reason why jettisoning is undesirable is that material is lost which could potentially be converted into oxygen, water, and possibly even sustenance. It is of vital importance that the maximum use and reuse be made of every pound of payload lifted from the surface of the Earth.

Jettisoning is the most obvious example of an open cycle system for space sanitation. Sewage and waste disposal techniques now employed in treatment plants on Earth must also be considered open cycle. Methods of waste disposal involving primary sedimentation, followed by an activated sludge process or trickling filter, and then by another process of sedimentation, possess a number of obvious disadvantages for use in space. The weight, bulk, and time required for the process to be completed would impose prohibitive limitations. Sedimentation would not be possible under zero gravity conditions, and as previously mentioned the aeration process would require much modification. The use of bacteria as a means of sewage treatment is not entirely satisfactory because the bacteria require oxygen, which is at a premium in the capsule, and give off carbon dioxide. Man alone respirates enough carbon dioxide to create a toxic atmosphere within the capsule in a short period of time. An ideal system is one that would absorb carbon dioxide and give out breathable oxygen.

It is possible that new techniques for handling terrestrial waste may have characteristics which are adaptable to space use. For instance, the Zimmerman process of wet combustion of waste by high pressure oxidation is much quicker than other methods and produces highly desirable end products, but it has disadvantages which would preclude its use in space. As used on Earth this process requires a pressure of 1,200 psi and attains a temperature of 460°F. The structural weight of the container for such a pressure would be excessive, and dissipation of excess heat is already a problem.

Semi-Open Cycle Systems

It has been shown by the Manhigh flights and by numerous ground tests that chemical absorption of waste gases is a workable solution, even though the weight penalty is prohibitive for flights of more than a few days' duration. It is possible that semi-open cycle systems might be devised in which the chemical absorbents can be used again after appropriate processing, and thus possibly reduce the total weight. However, the major inadequacy of all chemical systems is that they fail to provide the oxygen, water, or sustenance that is so vitally needed.

The ideal environment-control system is one in which the required oxygen and other supplies are obtained from waste products which are continuously converted to usable forms. There are several possible approaches to reuse of all waste material. One method would be to treat various wastes separately, i.e., having one system to maintain a breathable atmosphere, another to obtain fresh water from liquid wastes, and so on. Much work has already been done on such

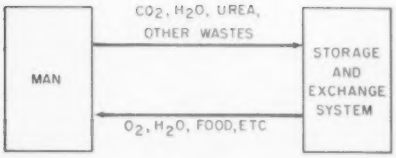

FIGURE 3.—Schematic closed ecology.

component recycling systems. For example, methods have been developed to purify urine by distillation, freezing, and electro-osmosis. These systems for recycling water can limit the weight of water required per man to 5 to 10 lb for an indefinite time. However, each of these methods has its own waste residue which must either be stored, jettisoned, or—ideally—reused.

Closed Cycle Systems

This brings about the consideration of a true closed ecology, in which essentially all waste products—solid, liquid, and gaseous—are used to provide fresh supplies needed by man. This concept is illustrated in Figure 3. On the surface of Earth, man interacts with the sum total of his environment, so that the right-hand box in the figure represents the atmosphere, the sun, all other living organisms (including his fellow men), the products of civilization, etc. In the space capsule, the problem is simpler, since the factors affecting the welfare of man are much less diverse, are known, and can be controlled.

Figure 4 is a schematic representation of a typical closed ecology for space applications. As shown, water, food, and oxygen are the basic supplies required by the man, who in turn produces water (vapor and liquid), carbon dioxide, urea, and other waste products. These materials are circulated through a filtering system, where, as on Earth, the wastes are processed into forms more easily handled in later phases of the cycle. Here also, the liquid wastes are processed to provide potable water for the man. After being processed in the filter chamber, the waste solution is piped into the algae tank where, by photosynthesis, the carbon dioxide is converted to oxygen. A light source is required for the photosynthetic process. Furthermore, the algae is a source of not particularly palatable, but still nourishing food. In all probability, some small amount of additional food will have to be supplied, and some small amount of unusable waste will have to be stored or jettisoned. This system is, however, essentially self-sustaining, and experiments with such a system at the Air Force School of Aviation Medicine

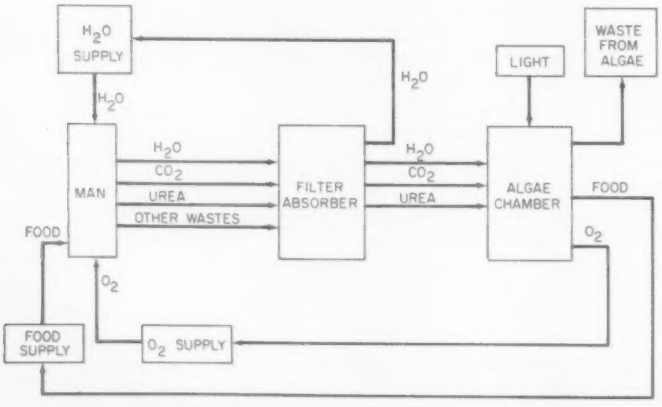

FIGURE 4.—Schematic closed ecology for space.

have been successful in supporting the life of mice for a number of days (3). The duration of these experiments was limited by operational failures of the equipment, not by inadequate production of required supplies.

How much algae is required to support a man? Until quite recently, the rule-of-thumb number was five pounds of algae per man, in a solution of one cubic meter of water, exposed to light in a sheet not more than about two inches deep. This requirement added up to more than a ton per man, exclusive of power requirements for lights and pumping. This discouraging outlook has recently been improved by the announcement of a new strain of algae, developed at the University of Texas, which increases 1,000 times a day, instead of only 8 times a day for the old strain (4). Similarly, scientists at the University of California at Berkeley have announced a particularly sturdy strain of algae capable of withstanding the thermal stresses anticipated in space flight (5). Thirty to forty gallons of algae solution of this type is sufficient to support a man, and it is anticipated that this figure will soon be reduced to 15 gal or about 125 lb/man.

This type of major technological breakthrough is essential if the weight of a closed ecology is to be kept within reasonable bounds. Certainly there is still room for improvement in the algae chamber design, and—perhaps even more—in the filter and processing unit. Unfortunately, there is a limit to the improvement possible in the third major component, the man, although some very interesting work is being done at the University of California at Los Angeles in quick-freezing of living subjects (to date non-human), which would reduce not only metabolic input requirements but also production of wastes.

Selection of Life-Support Systems

It should not be concluded, however, that even with major improvements, the closed cycle system will ultimately be used for all space missions. For short flights, such as ferrying trips to

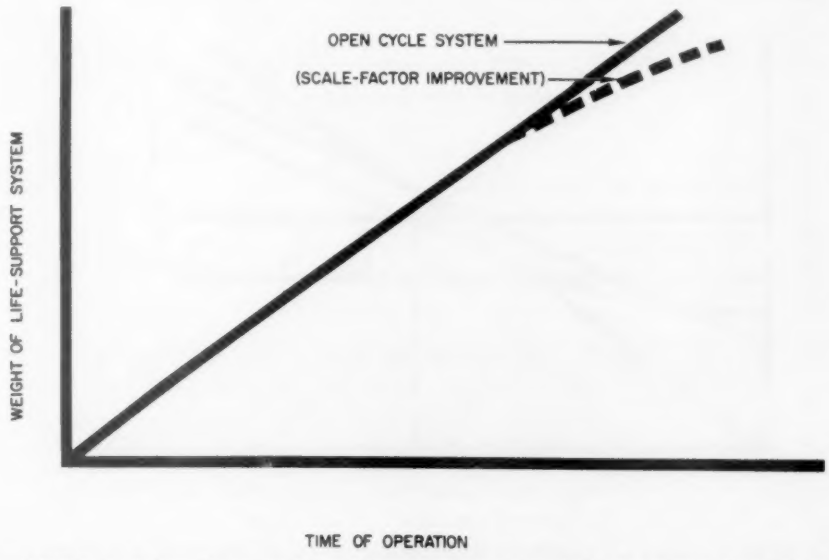

FIGURE 5.—Schematic comparison of life-support system weights for open cycle systems.

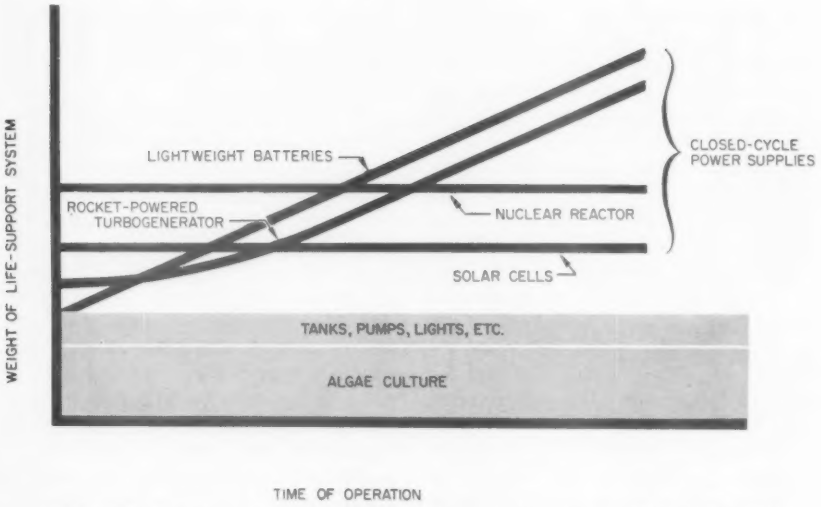

FIGURE 6.—Schematic comparison of life-support system weights for closed cycle systems.

permanent space stations, the open cycle system will probably suffice, so that the handling of waste products can be solved by simple storage for later disposal. The optimum solution for each mission must be determined independently, and will be a function both of the duration of the mission and of the technological state.

As shown in Figure 5, the weight of the open cycle system increases linearly with flight duration, with some possible improvements due to scale factor effects for long missions.

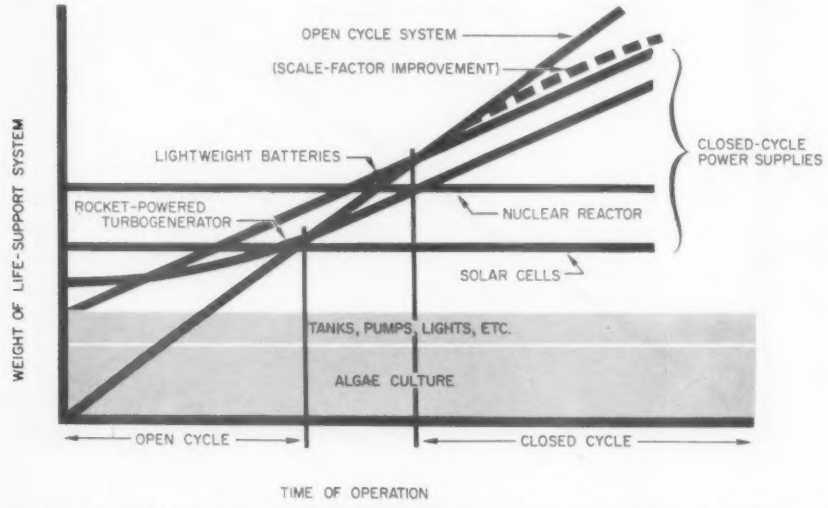

FIGURE 7.—Schematic comparison of life-support system weights, showing a combination of open cycle and closed cycle systems.

A similar plot for closed cycle systems is shown in Figure 6. The weights of the algae chamber and of the filter system, pumps, and piping are independent of flight time, the only variable factor being the power supply. The relative component weights of four different power supplies are indicated as additional weight increments above the fixed component values.

These two plots are combined in Figure 7, a qualitative example of the kind of study necessary to select an optimum life-support system for a given mission. It can be seen that, for some short flights, the open cycle system has the smallest weight penalty, while the closed cycle system is optimum for long duration missions. For flights of intermediate duration, the choice of life-support system is dependent primarily on the type of power supply which is available.

Summary

At present, vehicle capability limits the permissible weight for life-support systems to values so small that the open cycle system is quite adequate. However, the crossover point is within reach, not only through development of larger vehicles but through steady improvement in the efficiency of auxiliary power supplies. As space technology advances toward more sophisticated explorations, the duration of these manned missions will depend primarily on the fixed weight components of the closed cycle life-support system. This is an area in which the techniques of waste handling, as used on Earth, can be adapted to make a significant contribution to the ultimate exploration of space.

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AMINO ACIDS IN SEWAGE SLUDGES

By P. V. R. Subrahmanyam, C. Anandeswara Sastry, A. V. S. Prabhakara Rao, and S. C. Pillai

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Activated sludge was reported to contain appreciable amounts of practically all the essential amino acids (1), and one pound of solids from activated sludge had the same amount and kind of amino acids as one-half gallon of fresh cow's milk (2). Following these and other observations (3) further work was carried out, apart from a study of the quality of sewage effluents in relation to amino acids (4). The work described in this paper relates to a comparative study of the amino acid composition of activated sludge and other sludges, the rate at which the amino acids from sewage are removed by activated sludge, and the rate at which they are removed during the natural process of purification of flowing sewage at Bangalore.

Materials and Methods

Raw Sewage Solids

Representative samples of domestic sewage were collected and dried over a water bath and the dried materials were analyzed. This was done every time samples of sludge from the connected septic tanks were analyzed or when the sewage was used for the preparation of activated sludge. Similarly, samples of town sewage discharged at the outfalls and samples of the flowing sewage at various points were collected and dried for analysis.

Chemically Precipitated Sludges

Five-liter samples of sewage were treated with alum (450 mg/l) or lime (900 mg/l) and the sludges formed were removed and dried for examination.

Sludge from Septic Tanks

Samples of sludge from septic tanks were collected and dried. A sample of septic sludge kept in a bottle for about two years was also analyzed along with other samples.

Activated Sludge

Activated sludge was prepared in the laboratory by aerating successive portions of raw sewage in a bottle of 15-l capacity for a period of about one month (by the fill-and-draw method). Solids from the corresponding samples of raw sewage were also dried and analyzed along with the resultant activated sludge. Similar activated sludge was used for other experiments.

Protozoa Occurring in Activated Sludge

The whitish fluffy masses of *Epistylis* sp. (a peritrichous ciliate protozoan) occurring abundantly in the activated sludge tank (at Ambasamudram, Madras State) were repeatedly washed in distilled water, acetone-dried, and analyzed. A similar material was prepared from *Epistylis* sp. that developed in the activated sludge produced in the laboratory and this was also analyzed.

Other Materials

A variety of waste materials at varying stages of decomposition and other

TABLE I.—Nitrogen Content of Different Sewage Sludges and Other Materials

Material	Nitrogen (Percentage on oven-dry basis)
Raw sewage solids	2.96
Sludge precipitated with alum	2.86
Sludge precipitated with lime	1.15
Sludge from the septic tank	3.80
Activated sludge	7.73
Protozoa in activated sludge (*Epistylis* sp.)*	10.51
Cow dung	0.99

* Obtained from the activated sludge plant at Ambasamudram (Madras State).

washings may enter the sewerage system or may find their way into the flowing sewage at Bangalore. Because of this circumstance it was considered desirable to analyze a sample of cow dung for its amino acid content, for purposes of comparison.

The nitrogen content of the above mentioned materials are given in Table I.

Analytical Methods

The amino acids in the sewage sludges and other materials were determined by the circular paper chromatographic technique. The details of this technique, including the procedure for the preparation of the materials for the analysis of the amino acids, total and in the free form, are given in an earlier paper (4). The methods employed for assessing the quality of the sewage effluents were mostly those recommended by Theroux *et al.* (5) and "Standard Methods" (6).

Experimental Results

Amino Acids

The sludges and other materials listed in Table I were analyzed for their total amino acids and the results are given in Table II. Further studies on the amino acids (in the free form and total) in activated sludge and septic sludge were carried out, and the relevant observations are recorded in Table III.

Rate of Removal of Amino Acids by Activated Sludge

In a series of suitable flasks (4-l capacity) 1,850-ml samples of raw sewage were taken, 150-ml samples of activated sludge (15-per cent) were added, and air was bubbled through the suspensions. After aeration for one-half hour, one of the flasks in the series was removed, its contents allowed to settle, and the supernatant liquid

TABLE II.—Amino Acids in Sewage, Sludges, and Related Materials, mg/g of the Dried Solids

Amino Acid	Raw Sewage	Alum Sludge	Lime Sludge	Septic Tank Sludge	Activated Sludge	Protozoa (*Epistylis* sp.)*	Cow Dung
Cystine	3.9	1.6	1.0	0	4.2	8.0	0
Lysine and histidine	5.5	5.5	7.5	4.2	22.4	31.6	1.7
Arginine	6.7	4.4	2.5	5.2	14.6	20.2	0.8
Serine, glycine, and aspartic acid	18.3	7.3	6.3	9.7	31.4	34.6	1.7
Threonine and glutamic acid	9.5	10.0	7.0	8.8	40.1	46.1	1.7
Alanine	7.0	5.8	3.9	7.4	23.4	27.8	1.3
Proline	0	0	0	0	trace	trace	0
Tyrosine	3.5	4.4	4.6	3.4	10.8	13.1	0
Methionine and valine	15.1	8.7	3.2	7.8	27.8	37.8	2.3
Phenylalanine	11.1	5.5	5.9	4.1	26.5	30.1	2.3
Leucines	10.7	8.9	5.2	9.2	29.9	40.0	2.6
Total	91.3	62.1	47.1	59.8	231.1	289.3	14.4

* Obtained from the activated sludge plant at Ambasamudram (Madras State).

TABLE III.—Amino Acids in Sewage, Activated Sludge, and Septic Tank Sludges, mg/g of the Dried Solids

Amino Acid	Raw Sewage		Activated Sludge		Septic Tank Sludge		Septic Tank Sludge Kept for About 2 Yr	
	Free Form	Acid Hydrolysate	Free Form	Acid Hydrolysate	Free Form	Acid Hydrolysate	Free Form	Acid Hydrolysate
Cystine	trace	4.2	0.06	14.8	0	0	0	0
Lysine and histidine	trace	6.9	0.42	26.9	0.03	3.2	0.02	2.8
Arginine	trace	6.1	0.34	18.1	0.07	4.8	0	4.6
Serine, glycine, and aspartic acid	0.06	12.9	1.82	36.4	0.08	8.8	0.03	7.0
Threonine and glutamic acid	0.08	13.8	0.84	34.3	0.10	4.2	0.02	3.6
Alanine	0.04	7.3	0.90	22.1	0.07	4.8	0.02	2.6
Proline	0	0	trace	trace	0	0	0	0
Tyrosine	0.09	5.3	0.62	12.8	0	trace	0	trace
Methionine and valine	0.17	10.9	1.14	31.7	0.23	6.8	0.05	4.0
Phenylalanine	0.08	14.4	0.72	27.5	0.25	6.0	0.03	1.8
Leucines	0.21	6.6	1.26	29.9	0.22	2.2	0.02	1.4
Total	0.73	88.4	8.12	254.5	1.05	40.8	0.19	27.8

was collected by decanting through clean cotton wool. A portion of the supernatant liquid was immediately submitted to chemical and bacterial analysis in order to determine the extent of clarification and oxidation of the sewage. The bulk of the liquid was evaporated on a water bath and the dried solids used for the analysis of amino acids. Thus, after aeration of the mixture of sewage and activated sludge for further periods of 1, 2, 4, 6, and 24 hr, the supernatant liquids from the successive flasks in the series

TABLE IV.—Rate of Clarification and Oxidation of Sewage by Activated Sludge, mg/l*

Determination	Raw Sewage at Start	Effluents Obtained After Aeration of Sewage with 15-Per Cent Activated Sludge for Various Times					
		¼ hr	1 hr	2 hr	4 hr	6 hr	24 hr
Turbidity†	118	64	53	42	24	14	12
3-min oxygen absorption	21	11	10	8	4	3	2
4-hr oxygen absorption	74	40	34	26	16	14	11
BOD	201	156	129	92	24	16	9
DO	2.4	5.0	6.4	7.0	7.0	8.0	8.4
Total bacteria on nutrient agar (million/ml)	25.0	12.3	10.9	7.6	4.8	0.6	0.02
Ammoniacal nitrogen (N)	30.4	26.4	24.0	16.4	7.4	4.2	2.5
Albuminoid nitrogen (N)	7.0	3.8	3.3	2.5	1.0	0.9	0.3
Nitrite nitrogen (N)	0	trace	trace	1.0	1.7	trace	0
Nitrate nitrogen (N)	0	0	0	6.1	17.6	22.3	28.6
Nitrogen (N) in the dried solids from the effluents (per cent)‡	2.49	1.78	1.39	0.77	0.26	0.20	—

* Except as noted.

† Values obtained by using Klett Summerson colorimeter with 420 filter.

‡ Nitrite and nitrate nitrogen are not included.

TABLE V.—Rate of Removal of Amino Acids from Sewage by Activated Sludge, mg/g of the Dried Solids

Amino Acid	Raw Sewage	Effluents Obtained After Aeration of Sewage with 15-Per Cent Activated Sludge for Various Times*			
		¼ hr	1 hr	2 hr	4 hr
Cystine	9.4	1.7	0.8	0	0
Lysine and histidine	7.2	4.7	4.6	2.3	0
Arginine	11.0	8.6	7.6	3.5	0
Serine, glycine, and aspartic acid	18.3	14.2	13.9	9.5	trace
Threonine and glutamic acid	4.5	3.1	2.0	1.4	trace
Alanine	6.1	4.2	4.0	3.0	0
Proline	0	0	0	0	0
Tyrosine	3.5	1.8	1.8	trace	0
Methionine and valine	10.0	7.4	5.2	2.9	0
Phenylalanine	4.7	3.3	1.6	0.4	0
Leucines	11.3	6.9	6.8	2.8	0
Total	86.0	55.9	48.3	25.8	—

* Effluents obtained after aeration for 6 and 24 hr did not contain any amino acid.

were removed and analyzed. Results obtained from three runs were similar. One set of results relating to the quality of the effluents and their amino acids at different stages of purification is presented in Tables IV and V.

Similar experiments were conducted by adding chemicals such as copper sulfate (20 to 50 mg/l) with a particular view to rendering the dominant protozoa (species of *Epistylis* and other vorticellids) in the activated sludge inactive or selectively killing them and using the partially sterilized

TABLE VI.—Clarification and Oxidation of Sewage Flowing on Land Having 1-in-50 Gradient, mg/l*

Determination	At the Outfall	Distance in Miles from the Outfall					
		0.17	0.38	0.67	0.71	0.92	1.29
Turbidity†	198	134	100	92	50	30	12
Total solids	1,164	1,023	960	939	892	887	809
3-min oxygen absorption	38.0	24.2	20.8	15.0	9.0	3.8	2.6
4-hr oxygen absorption	75.6	50.8	30.4	20.0	16.8	10.0	6.4
BOD	254	168	142	121	56	32	10
DO	0.4	1.6	2.8	4.0	6.4	7.6	7.8
Total bacteria on nutrient agar (million/ml)	38.0	21.0	18.0	11.5	—	0.01	0.002
Ammoniacal nitrogen (N)	36.4	25.2	20.8	18.8	16.2	13.2	6.4
Albuminoid nitrogen (N)	18.7	12.3	10.9	8.4	3.6	1.9	0.7
Nitrite nitrogen (N)	0	0	0	0	trace	0.45	0.22
Nitrate nitrogen (N)	0	0	0	0	0.02	0.37	0.63
Total nitrogen (N)	65.2	41.8	34.0	28.6	21.6	16.8	8.9
Nitrogen (N) in the dried solids from the effluent (per cent)‡	2.54	1.53	1.30	0.90	0.56	0.34	0.18

* Except as noted.

† Values obtained by using Klett-Summerson colorimeter with 420 filter.

‡ Nitrite and nitrate nitrogen not included.

TABLE VII.—Removal of Amino Acids from the Sewage Flowing on Land Having 1-in-50 Gradient, mg/g of the Dried Solids

Amino Acid	At the Outfall	Distance in Miles from the Outfall					
		0.17	0.38	0.67	0.71	0.92	1.29
Cystine	6.9	3.2	3.1	2.5	1.4	trace	0
Lysine and histidine	9.4	4.3	4.0	2.4	0.9	trace	0
Arginine	6.1	3.5	3.0	2.1	0.8	trace	0
Serine, glycine, and aspartic acid	17.8	10.2	9.7	7.7	3.4	1.2	trace
Threonine and glutamic acid	26.5	16.7	14.9	10.2	4.1	1.8	trace
Alanine	9.4	3.5	2.3	2.0	1.2	0.5	trace
Proline	0	0	0	0	0	0	0
Tyrosine	5.3	trace	trace	trace	trace	0	0
Methionine and valine	11.7	6.9	4.9	2.8	1.2	0.8	0
Phenylalanine	8.3	5.4	5.0	2.9	trace	trace	trace
Leucines	13.9	6.3	4.6	2.9	1.3	trace	0
Total	115.3	60.0	51.5	35.5	14.3	4.3	—

sludge for the treatment of the sewage. The results indicated that the rate of removal of the amino acids from the sewage, as well as that of its purification, by the sludge containing inactive or dead protozoa was comparatively very poor.

Rate of Removal of Amino Acids during Natural Purification

It was reported earlier that domestic sewage flowing in the three natural channels at Bangalore was purified and was practically free from amino acids, although the distance that the sewage had to cover for its purification and removal of the amino acids depended on the gradient of the channel (4). The channel having 1-in-50 gradient was later selected for a more detailed study of the amino acids at various points in the channel. The sampling points and the observations repeatedly made on the extent of purification and removal of amino acids at those points are given in Tables VI and VII.

Discussion

Activated sludge contained a much higher amount of amino acids than chemically precipitated sludges, septic tank sludge, raw sewage solids, or cow dung. This feature of activated sludge may, at any rate, partly be due to its action of rapidly removing practically all the amino acids from the sewage.

It was rather significant to observe that by mere vigorous flow of raw sewage in the natural channel at Bangalore the amino acids in the sewage were removed as rapidly as by activated sludge. The experiments with activated sludge and the observations on the flowing sewage showed that the removal of amino acids accompanied or formed a part of the process of purification of the sewage. The amino acid content of a treated sewage or purified effluent was found to influence its quality or stability (4).

The observations bearing on the clarification of sewage with chemicals such as lime and alum raise the problem of dissolved putrescible matter in the medium. Among the known methods of sewage treatment, activated sludge appeared to be the only system capable of removing the amino acids practically entirely from sewage. The effluent from the septic tank contained an appreciable amount of amino acids (4), and the amount of amino

acids retained in the sludge in the septic tank was even less than that in the raw sewage solids. The samples of septic sludge analyzed did not show the presence of cystine. These and other changes in the septic tank are apparently brought about by bacteria and other forms of life which can function under anaerobic or near-anaerobic conditions. The conditions in the activated sludge tank are, however, very different.

In discussing the several features characteristic of the activated sludge process, Baly (7) stated that "the rapidity of the process is most remarkable when it is remembered that the hydrolysis and oxidation of the harmful impurities by bacterial action are relatively slow, even under the optimum conditions of oxygen supply," and that "the process not only purifies the sewage from all harmful impurities, but also removes the bacteria, so that a sterile or almost sterile effluent is obtained." In this connection, the influence of certain protozoa (such as the species of *Epistylis*) on the color, composition, and activity of activated sludge was studied (8).

The dominant protozoa (*Epistylis* sp.) in the activated sludge, on equal dry basis, however, contained even a higher amount of amino acids than the sludge. Similar protozoa were found to occur abundantly in the region of natural purification of flowing sewage (9). Inactivity or death of the protozoa in activated sludge adversely affected the removal of amino acids from sewage by the sludge. Investigation into the requirements of amino acids for the growth of these protozoa and other organisms would be interesting. Mention may be made of the study of the relation of the concentration of amino acids with the growth of *Sphaerotilus* which developed only in diluted sewage (10).

The observations recorded in this paper, however, emphasize the question of dissolved putrescible matter like amino acids in sewage and show more clearly an aspect or principle of purification which has not been adequately appreciated in activated sludge and which is also appreciable in the natural purification of flowing sewage. This principle seems to have a marked bearing on the conservation or concentration of nitrogen in the system.

Summary

The amino acid composition of raw sewage solids, chemically precipitated sludges, septic tank sludge, activated sludge, protozoa in the activated sludge, and cow dung was studied. All these materials contained practically all the essential amino acids, the activated sludge and the protozoa containing by far the highest amounts. Cystine was not present in the samples of septic tank sludge and cow dung examined.

During purification of sewage by the activated sludge process as well as during natural purification of flowing sewage, the amino acids in the sewages were removed at a rapid rate, so that the final effluents from these systems were practically free from the amino acids. The significance of these and other observations is discussed.

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WATER WORKS OFFICERS

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EFFECTS OF SURFACE ACTIVE AGENTS ON BUBBLE AERATION

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Aeration processes involve a two-phase system. The rate of gas transfer in this system depends on the gas-liquid interface—its total area and its chemical, physical, and mechanical character. With small concentrations of surface active agents in solution, their presence at the interface markedly reduces the rate of gas transfer. The rate is reduced with increasing concentration up to a certain critical concentration, above which the rate is unaffected. In aeration bubbling processes, at high concentrations of certain surface active compounds, the total interfacial area may be increased, resulting in recovery of the over-all rate of gas transfer.

This study endeavors to analyze theoretically and experimentally the various factors involved in the effect of surface active agents on the rate of oxygen transfer in aeration processes. A more complete exposition of this study can be found in a report prepared at the University of North Carolina (1).

Previous Results

The effect of synthetic surface active agents on aeration efficiency in sewage and industrial waste treatment processes has been widely reported. Sawyer (2) and Lynch and Sawyer (3) state that surface active agents lower the rate of gas transfer in aeration processes. Eckenfelder (4), in studies on the rate of oxygen absorption in various industrial wastes, mentions the effect of soap solutions on lowering the aeration rate. Ippen *et al.* (5), Ippen and Carver (6), and Carver (7) state that in aeration bubbling processes a surface active agent lowers the rate of oxygen absorption; however, in some instances, despite this lowering effect, the rate of absorption of oxygen was found to be greater in solutions containing surface active agents because of the increase of the surface area afforded by the smaller bubbles. Downing *et al.* (8) (9) observed that the reduced rate of oxygen absorption caused by anionic household detergents depended on the detergent concentration, the method of aeration, and the quantity of suspended solids present in the activated sludge process. It was also reported by the Water Pollution Research Board (10) and Gaden and Hixon (11) that surface active agents, at one specific concentration, lowered the rate of oxygen absorption in the reaeration of flowing water. The physicochemical properties of the gas-liquid interface as the governing factor on the rate of gas transfer have not been elucidated. Recently, Gaden (12), in aeration studies on biological systems, showed that there is a tendency for recovery of the oxygen transfer coefficient on increasing the surface active agent concentration after an initial decrease.

In summary, many investigators have found that the effect of surface active agents is to lower the rate of gas transfer in aeration processes. Except for Gaden (12), however, none of the investigators studied this effect at surface active agent concentrations high enough to increase the over-all oxygen transfer. Other studies (13) (14) (15)

(16) (17) in the field of surface chemistry and biology discuss the effect of surface adsorption at the gas-liquid interface on the rate of gas transfer.

Basic Theoretical Considerations

Effect of Surface Active Agents on the Physicochemical Properties of the Gas-Liquid Interface

The gas-liquid interface is characterized by the fact that the concentration of any dissolved component is much greater at the interface than in either adjacent phase (13) (14) (15) (16) (17). In the two-phase system (air-water), the molecules of a soluble component exhibit a higher concentration at the interface than in either water or air. Such molecules pass through the surface of water and they thicken the interfacial region, thus reducing the interfacial tension.

Generally, the physicochemical character of the interfacial adsorbed film of surface active agent in air-water systems depends on:

1. Types of surface active agent used.
2. Number of carbon atoms in the adsorbed molecule.
3. Molecular configuration.
4. The mode of adsorption and orientation.
5. The compressibility and spreading of the adsorbed film.
6. The presence or absence of hydrogen bonding either in the adsorbed film itself or between the polar groups in the film and molecules of water.

Surface-adsorbed films are usually classified into three groups: gaseous, liquid-expanded, and condensed films. Soluble surface active agents form monomolecular surface adsorbed films of the gaseous type.

Surface Adsorption and the Critical Micelle Concentration (CMC)

Upon considering the thermodynamic properties of a solution of a surface active agent, one of the most striking colligative properties of the solution is the critical micelle concentration. This is the concentration at which the solution forms colloidal aggregates by the association of molecules or ions. These aggregates are in reversible thermodynamic equilibrium with the surrounding environment. Below this concentration the properties of the solution are similar to those of ordinary electrolytes, but above this concentration, the solution behaves differently in that changes in the properties of the solution are much less marked with a change in concentration. Surface tension, conductivity, freezing point, viscosity, density, transference number, and refractive index have been cited as examples of properties that show abrupt changes in slope at the critical micelle concentration. Presumably, this change in slope must occur over a concentration interval, but this interval is fairly narrow. For practical purposes, it is legitimate to speak of a critical micelle concentration rather than a critical micelle region (13) (18) (19).

Basic Equation to Calculate the Gas Transfer Coefficient

The general equation for gas transfer, as developed from Fick's first law, can be written as follows:

$$\frac{dC_t}{dt} = K_L\left(\frac{A}{V}\right)(S - C_t)$$

$$= K_L a(S - C_t) \ldots (1)$$

where

C_t = concentration of the gas at time t, mg/l;

S = saturation concentration with respect to the gas, mg/l;

$K_L a$ = the gas transfer coefficient, $\text{min}^{-1} = K_L\left(\frac{A}{V}\right)$; and

K_L = the absolute gas transfer coefficient, cm/min.

The integrated equation between t_1 and t_2 becomes

$$\log (S - C_{t_2}) - \log (S - C_{t_1}) = -K_L \left(\frac{A}{V}\right)(t_2 - t_1) \quad (2)$$

This can be represented as a straight line with the slope representing the rate of gas transfer

$$K_L\left(\frac{A}{V}\right) = K_L a = -\frac{\Delta \log (S - C_t)}{\Delta t} \quad (3)$$

Experimental

The apparatus used in measuring the rate of oxygen transfer is shown in Figure 1 and is described below. The development of the continuous recording system is described elsewhere (20).

Aeration Column

The aeration column is a glass cylinder with a capacity of 1.5 l. The cylinder stopper at the bottom has openings for the oxygenation capillary, for a nitrogen diffuser used to sweep oxygen from the solution, and a third opening serves as an outlet for the water in the column. The inlet is near the top of the column. A continuous sample is withdrawn by a centrifugal pump from the bottom outlet and is circulated at a constant rate through the polarographic cell and back to the aeration chamber through the upper inlet.

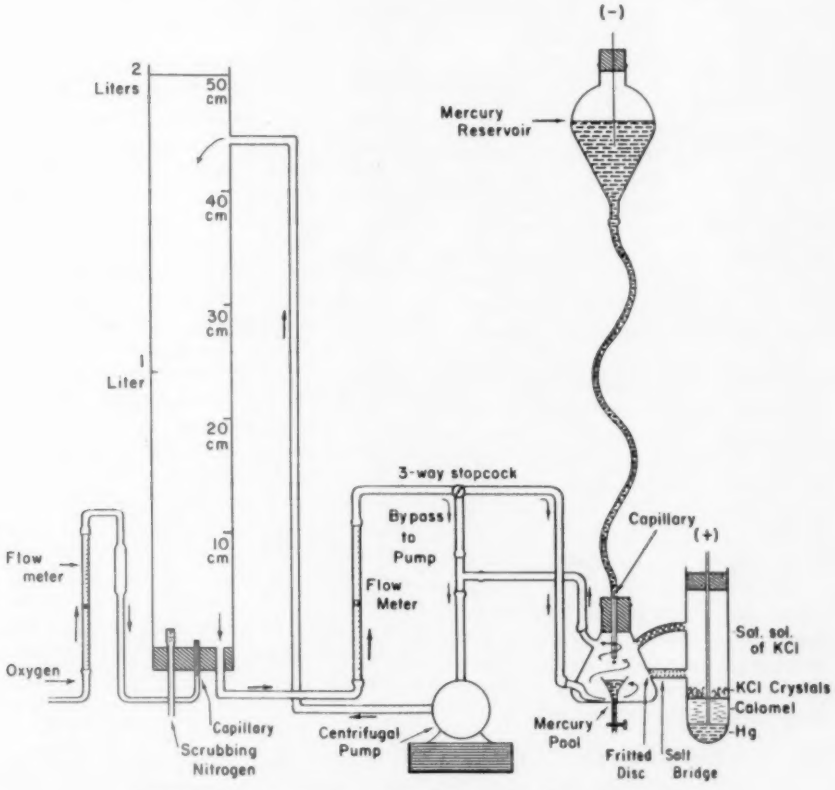

FIGURE 1.—Apparatus for studying rate of oxygen transfer.

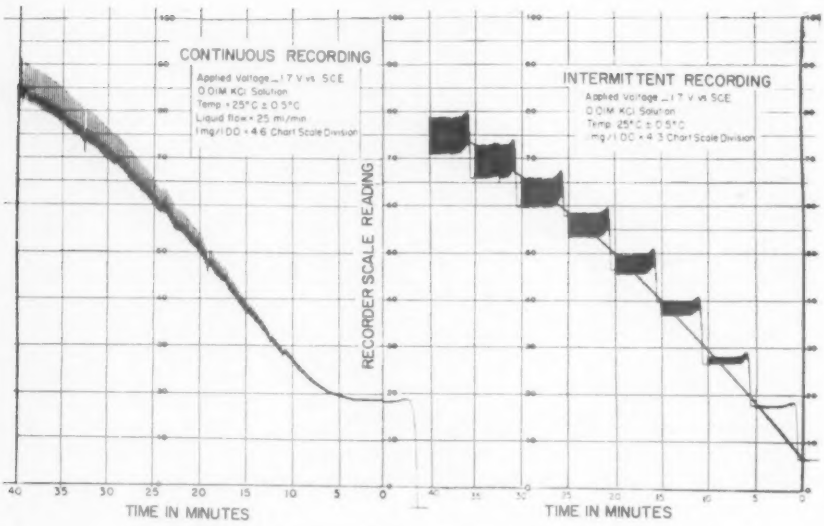
CONTINUOUS RECORDING
Applied Voltage -1.7 V vs. SCE
0.01M KCl Solution
Temp. = 25°C ± 0.5°C
Liquid flow = 25 ml/min
1 mg/l DO = 4.6 Chart Scale Division
INTERMITTENT RECORDING
Applied Voltage -1.7 V vs SCE
0.01M KCl Solution
Temp. 25°C ± 0.5°C
1 mg/l DO = 4.3 Chart Scale Division
RECORDER SCALE READING
TIME IN MINUTES
TIME IN MINUTES

FIGURE 2.—Automatic recording of oxygen absorption.

Electrode System

A specially designed dropping mercury polarographic cell is connected to a saturated calomel electrode (anode) by means of an agar salt bridge through a side connection and a small fritted glass disc, which also acts as a support. The mercury capillary was under a pressure of 58 cm of mercury, providing a drop time of 2.8 sec and a rate of mercury flow of 1.8 mg/sec.

The dropping mercury cathode and the saturated calomel electrode were connected to a Sargent Manual Polarograph III. The applied voltage was set to impress across such a cell an emf value corresponding to the decomposition voltage of the second wave of oxygen. The use of the second oxygen wave was dictated by the need to avoid the addition of a maximum suppressor which is required when the first wave is used and by the fact that anionic surface active agents interfere with the electrode kinetics at the first oxygen reduction wave (20).

Amplifier and Recording Unit

The minute d-c current from the cell was passed through a specific resistance. The minute d-c voltage across this resistance was introduced into a continuously balanced Brown Electronik recorder. The record was on a Brown chart Number 5430N with a calibrated scale in terms of mv. The chart was driven at a regulated speed.

This system makes it possible to detect and record continuously the rate of oxygen transfer in the test solution without any change of the original volume as a result of sampling. Figure 2 shows recorded aeration curves with both continuous and intermittent recording.

To eliminate the errors due to gas transfer at the surface of the column, the tests were run using oxygen gas, beginning with the solution at DO concentrations slightly below saturation with respect to air and ending at concentrations slightly above. The rates were then determined from the semilogarithmic plots at the DO saturation value.

Material Used

The surface active agent used in this study was Aerosol O.T., a product of the American Cyanamid Company.

Its chemical and physical properties are summarized as follows:

1. Chemical name: sodium dioctyl sulfosuccinate
2. Structure:

$$\begin{array}{l} CH_2COOC_8H_{17} \\ | \\ CHCOOC_8H_{17} \\ | \\ SO_3Na \end{array}$$

3. Gram molecular weight: 444
4. Ionic nature: anionic
5. Number of carbon atoms in the non-polar group: C_{16}
6. Solubility in water at 30°C: 24 g/l

The behavior of Aerosol O.T. with regard to aeration has been shown to be similar to that of alkyl benzene sulfonate (ABS) which is the base of most household detergents (1).

Methods of Making Observations

1. Photographs of the bubbles at different levels in the column were used to study the mechanism of bubble formation, bubble volume, and bubble dynamics at various rates of oxygen flow and at varying concentrations of the surface active agent.

2. Surface tension measurements of the surface active agent solution were made by means of a Ceneo-DuNouy Tensiometer.

3. The critical micelle concentration of the Aerosol O.T. solution was determined by the surface tension of the

TABLE I.—Effects of Increasing Concentrations of Surface Active Agents on Bubble Aeration*

Conc. Aerosol O.T. (mg/l) 1.	Avg. Dist. between Bubbles (cm) 2.	No. of Bubbles in Col. 3.	Rate of Bubbling (bubbles/min) 4.	Time of Bubble Formation (sec) 5.	Vel. of Bubble Rise (cm/sec) 6.
0	8.55	6	141	0.376	22.70
10	7.75	6.5	141	0.356	21.75
20	7.72	7	149.1	0.355	21.70
40	7.47	7.5	149.3	0.352	21.23
80	6.94	8	160.6	0.352	19.75
160	4.72	11.5	226.2	0.242	19.45
320	3.45	15	—	0.200	17.42
640	2.45	22.5	—	0.147	16.67

Conc. Aerosol O.T. (mg/l) 7.	Bubble Volume (cc $\times 10^6$) 8.	Bubble Diam ($cm^2 \times 10^4$) 9.	Bubble Area ($cm^2 \times 10^4$) 10.	Total Surface (cm^2) 11.	K_La ($min^{-1} \times 10^2$) 12.	K_L ($cm\ min^{-1}$) 13.	Surface Tension (erg cm^{-2}) 14.
0	7.2	2.4	1.81	1.59	1.97	1.87	71
10	6.94	2.37	1.76	1.71	0.73	0.643	50.5
20	6.42	2.30	1.66	1.62	0.81	0.74	45.6
40	6.47	2.31	1.67	1.68	0.86	0.76	42.0
80	6.22	2.28	1.63	1.78	0.89	0.76	38.2
160	4.36	2.04	1.3	2.31	0.94	0.61	35
320	3.74	1.93	1.12	2.54	1.29	0.76	31
640	2.61	1.71	0.92	2.83	1.47	0.77	27

* Gas flow rate maintained constant at 1.2 cc/min; Volume of liquid, 1.5 l. Col. 2 and 3: Values obtained from photographs. Col. 4: Experimental observations. Col. 5: Calculated by obtaining the statistical mean of the time in seconds between the detachment of two successive bubbles from the capillary. Col. 6: Experimental observation.

Col. 8: $\frac{1.2 \times \text{Col. } 6}{\text{Col. } 3}$. Col. 9: $\left(\frac{\pi \times \text{Col. } 8}{6}\right)^{\frac{1}{2}}$. Col. 10: $\pi \times (\text{Col. } 9)^2$. Col. 11: Col. 10 × Col. 3.

Col. 12: From experimental data plotted in Figure 3. Col. 13: Col. 12 $\times \frac{1.5}{\text{Col. } 11}$.

solution. The rate of the decrease in surface tension with increasing concentrations of the surface active agent in solution shows an abrupt change which generally coincides with the CMC. The semilog plot of surface tension versus surface active agent concentration shows a sharp change in the slope at a concentration corresponding to the CMC (19).

Experimental Conditions

Because of the great number of factors that can affect the rate of oxygen transfer by bubble aeration, the following conditions were maintained constant during each test:

1. Hydrostatic head on the gas capillary,
2. Gas flow rate, and
3. Temperature—25°C ± 0.5°C.

Experimental Results

Effect of Surface Active Agents on the Rate of Gas Transfer

Data were collected at several oxygen flow rates. For purposes of illustration and analysis, the data collected at a rate of 1.2 cc/min are shown in Table I. The effect of varying concentrations of Aerosol O.T. on the over-all oxygen transfer coefficient K_La in bubbling aeration is shown in Figure 3. The values of the over-all transfer coefficient values were calculated in accordance with Equation 3. On increasing concentration of the surface active agent, the coefficient K_La drops to a minimum corresponding to a concentration of about 20 mg/l of Aerosol O.T. On further increase of the surface active agent concentration, there is a gradual recovery of K_La which is very slow at the beginning. The extent of the recovery depends on the oxygen flow rate; the higher the gas flow rate, the lower the recovery.

In the presence of 20 ml of paraffin oil at the top of the aeration column, the K_La drops to a lower minimum value after which there is no apparent recovery on further addition of the surface active agent. The initial reduction of the gas transfer coefficient was found to be 62 per cent in absence of paraffin oil and 75 per cent in presence of paraffin oil for the same oxygen flow rate.

When oxygen was bubbled through a fritted glass diffuser, the over-all oxygen transfer coefficient increased as the concentration of the surface active agent increased (Figure 3).

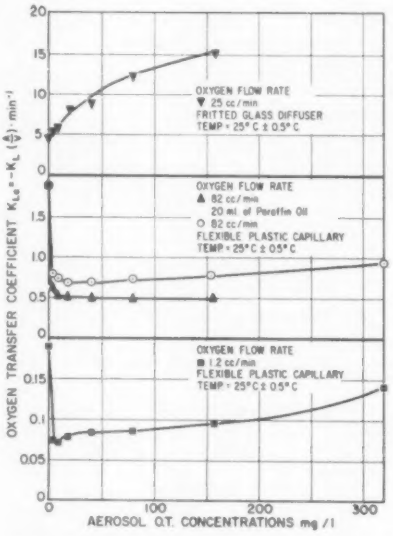

FIGURE 3.—Oxygen transfer coefficient at varying Aerosol O.T. concentrations. K_La values shown are actual values times 10^3.

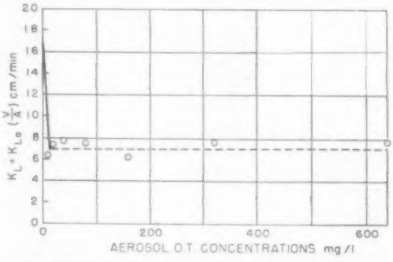

FIGURE 4.—Absolute oxygen transfer coefficient. K_L values shown are actual values times 10.

Figure 4 shows the absolute oxygen transfer coefficient, K_L, at varying Aerosol O.T. concentrations, calculated in accordance with Equation 3. This reaches a minimum in the range of from 20 to 40 mg/l, and thereafter it is relatively constant.

The Effect of Surface Active Agents on Bubble Formation and Bubble Dynamics

1. Bubble Formation.—The effect of Aerosol O.T. on the time of bubble formation is shown in Figure 5. On increasing the surface active agent concentration, the time of bubble formation decreases by 60 per cent in the concentration range of from 0 to 640 mg/l.

2. Bubble Volume.—With a constant rate of gas flow, the time of bubble formation is directly related to the bubble volume. The bubble volume, therefore, decreases on increasing the surface active agent concentration as shown in Figure 6. A 64-per cent decrease in the bubble volume was observed in the concentration range of from 0 to 640 mg/l of Aerosol O.T.

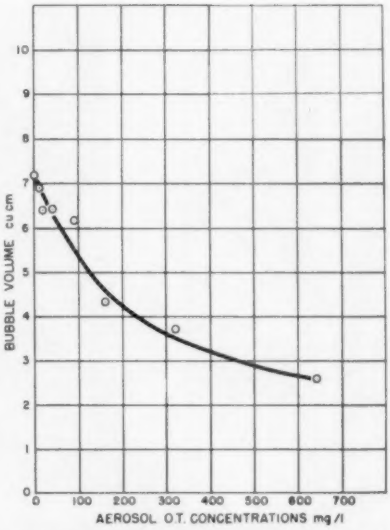

FIGURE 6.—Bubble volumes at varying Aerosol O.T. concentrations and an oxygen flow rate of 1.2 cc/min. Volume shown is actual volume times 10^5.

3. Bubble Dynamics.—The effect of Aerosol O.T., and, therefore, bubble size, on the velocity of bubble ascent is shown in Figure 7. As the bubble volume increases, the velocity of bubble ascent also increases.

Photographic studies, as illustrated in Figure 8, confirm that an increase in the surface active agent concentra-

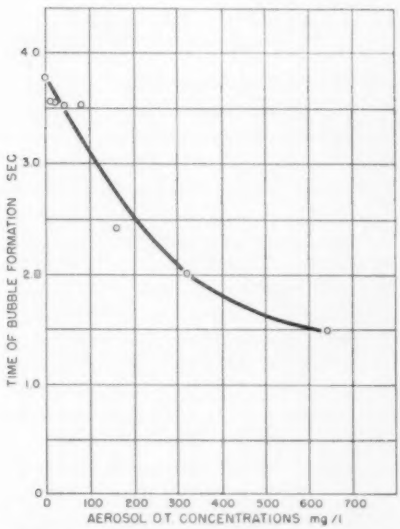

FIGURE 5.—Time of bubble formation at varying concentrations of Aerosol O.T. Time shown is actual time times 10.

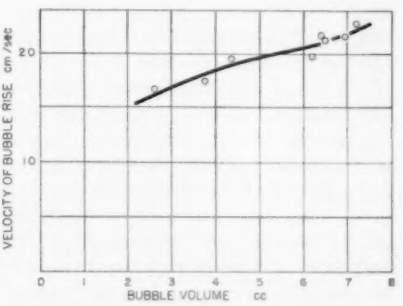

FIGURE 7.—Velocity of bubble as a function of bubble size. Volume shown is actual volume times 10^5.

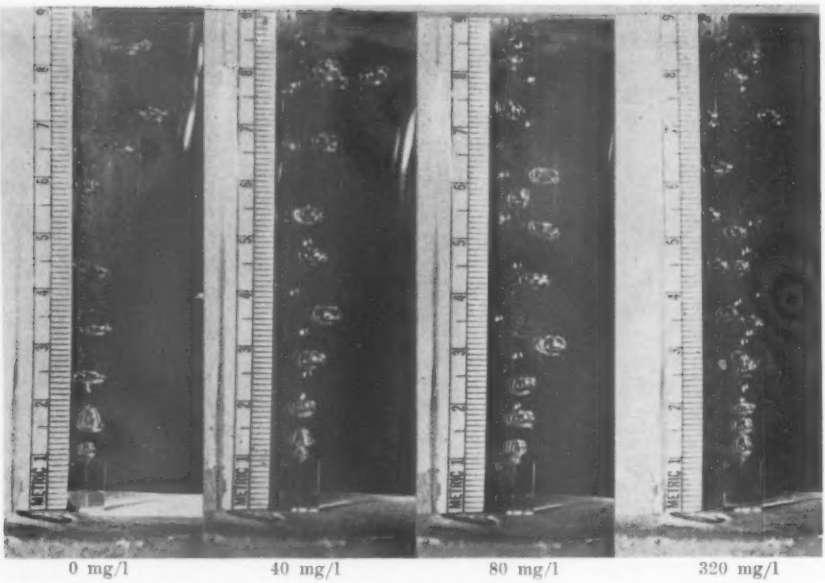

FIGURE 8.—Bubble formation and dynamics at varying Aerosol O.T. concentrations and an oxygen flow rate of 82 cc/min.

tion causes a higher rate of bubbling. The bubble on being detached from the surface of the capillary passes through various shapes. First it acquires a strawberry shape, then flattens into an elliptical shape with the long axis parallel to the liquid surface. This passes to another ellipsoid which is less flattened and the previous process is repeated. Actually, the bubble changes in shape until an equilibrium is attained after which the bubble travels at a steady state with a rather fixed shape. The photographs show that on the addition of Aerosol O.T. at constant gas flow, much smaller bubbles are produced at higher rates, the effect depending on the amount of surface active agent added. These bubbles are detached from the surface of the capillary with a lower velocity, and they travel at relatively low velocity. The bubble is relatively rigid and does not change much in shape during its ascent. At higher concentrations of the surface active agent, this effect is more prominent.

4. Total Exposed Surface Area.—Photographic studies show that the total exposed surface area increases

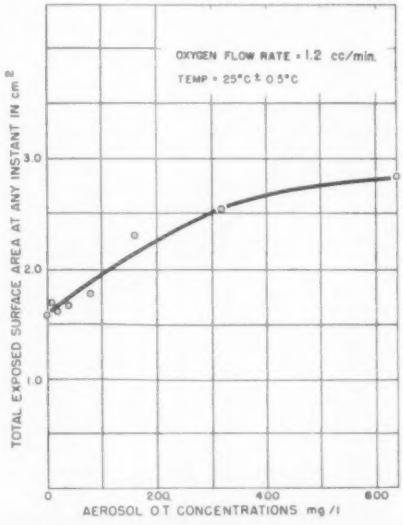

FIGURE 9.—Total exposed surface area of bubble.

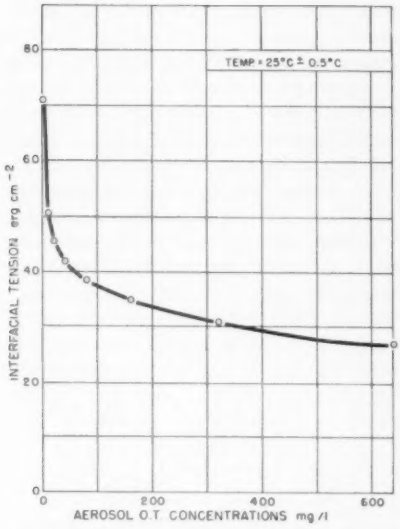

FIGURE 10.—Air-water interfacial tension at varying Aerosol O.T. concentrations.

with increasing Aerosol O.T. concentrations. Calculated values of the total exposed surface area are shown in Figure 9.

The Physicochemical Character of Aerosol O.T. in Solution

1. The Interfacial Tension at the Gas-Liquid Interface.—In a 0.01 M KCl solution, the presence of 40 mg/l of Aerosol O.T. resulted in a 41-per cent decrease in the surface tension at the gas-liquid interface as shown in Figure 10. On further addition of surface active agent, the interfacial tension decreased slowly and assumed a rather constant value at higher concentrations of the surface active agent. It is interesting to note the similarity between the effect of the surface active agent on the gas-liquid interfacial tension and the same effect on oxygen transfer coefficient, bubble formation, and bubble dynamics.

2. CMC Determinations.—In determining the CMC for Aerosol O.T. in 0.01 M KCl solution, the surface-tension method, possibly the most accurate, indicated a CMC value of 20 mg/l, which coincides with values reported in the literature (21).

Discussion of Effect of Surface Active Agents on Lowering the Rate of Gas Transfer

Mass transfer across the gas-liquid interface depends to a great extent on the physicochemical and mechanical properties of the phase boundary. In this two-phase system the following conditions are shown to occur:

1. In an air-water system, the sharp transition from one phase to another requires that the gas molecules have kinetic energy to cross this boundary at ordinary temperature; the amount of energy depends primarily on the liquid film thickness.

2. Virtually all mass transfer resistance is offered by the liquid surface film.

3. On adding a surface active agent, a higher concentration of this solute is produced at the interface, in the form of monomolecular layer of adsorbed surface active molecules.

4. The molecules in the adsorbed film acquire a certain orientation, depending on their structural pattern.

5. This film adds more material resistance to gas permeability, and in an aeration process the gas molecule will lose more of its kinetic energy in crossing the interface (16) (17).

Between the liquid and gas phases there exists a definite quantity of free energy associated with each unit of the interfacial area. This amount of surface energy is dependent on the adsorbed surface active film. On increasing the concentration of the surface active agent in solution, the lateral compression acquired by the newly arrived molecules to the surface adsorbed film reduces the apparent free energy prevalent at the interface. The force of compressibility in the adsorbed film increases, and the number of molecules per unit area of the inter-

face also increases. The area per molecule decreases as a linear function of compression until a certain value is reached at which the area per molecule stays more or less constant along with the surface tension. This was found to correspond with the CMC of the surface active agent solution (13–19). The constant value of the surface energy at concentrations above the CMC signifies a saturated film of uniform thickness. Actually, surface saturation at and above the CMC is dependent on many factors:

1. The rate of diffusion of the surface active agent molecules to the surface.
2. The rate of adsorption of the surface active agent molecules at the interface.
3. Agitation, stirring, and convection current effects usually lead to lower CMC values and by virtue of the mechanical transport, more surface active agent molecules reach the interface.
4. Presence of organic or inorganic salt impurities.

On examining the curve showing the rate of gas transfer at varying concentrations of Aerosol O.T. (Figure 3), the gas permeability of the interfacial adsorbed film decreases rapidly with increasing surface active agent concentration. It ceases to decrease at and above the CMC. In this region, the gaseous adsorbed film is most compact and has the highest force of compression, the lowest area per molecule, and the maximum thickness. The resistance attributable to the film thickness lowers the absolute oxygen transfer coefficient, K_L (Figure 4), by an amount depending on the mechanical properties of the surface film.

Working with a cationic surface active agent (15), a regular decrease of Henry's law constant was reported to be linear with the logarithm of the surface active agent concentration. Extrapolating the experimental straight line to the concentration of the surface active agent corresponding to the lowest value of Henry's law constant resulted in a minimum concentration corresponding to the CMC. Accordingly, maximum resistance to gas transfer due to surface active agent adsorption at the interface should occur at the CMC. Recent work (18) shows that not only surface adsorption of a surface active agent occurs as a monomolecular layer at low concentration, but also at higher concentrations the tracer method revealed for the first time the existence of multilayer adsorption under the interface, equivalent to 30 molecular layers in some cases. This signifies that at high concentrations the interface consists of an adsorbed charged monolayer which attracts an ionic atmosphere. Thus it appears that in an aeration system most of the resistance to diffusion of a gas molecule across the interface is offered by the monomolecular adsorbed film of the surface active material formed at low concentrations. At higher concentrations the adsorbed multilayers of micellar character offer only small contribution to the resistance of mass transfer across the interface. Accordingly, with increasing Aerosol O.T. concentrations, after an initial rapid decrease, the K_L value becomes more or less constant in the region of the CMC.

Discussion of Effect of Surface Active Agents on Increasing the Rate of Gas Transfer

In a bubbling process, bubble formation, volume, and dynamics depend on the following factors (23–28):

1. Bubble formation depends on the method and the device used for bubbling, time of bubble formation, material and texture of the surface of the aperture, shape of the aperture, arrangement of apertures, position of apertures, and the physicochemical conditions of the liquid phase.

2. Bubble shape and size depend on the mode of formation, height of the water column, rate of gas flow, the physicochemical conditions of the liquid phase, i.e., surface tension and to a lesser extent viscosity, specific gravity, and the presence of impurities.

3. Bubble dynamics are governed by all the previously listed factors. Rigid spherical bubbles and circulating spheroidal bubbles are two extremes of bubble conditions which are governed by bubble formation, bubble shape, size, age, and the physicochemical properties of the liquid phase.

Within the life span of a bubble, there are three different phases with three different rates of oxygen transfer. The first phase is the "bubble formation," during which time it is reported (6) (7) that there is a high rate of gas transfer due to the continual expansion of the fresh liquid gas interface, thus forming a steep oxygen concentration gradient across the bubble wall. The second phase is the "bubble ascent." During the bubble's journey to the surface, the rate of transfer acquires lower stable values until the bubble bursts at the surface. The third phase is the end of the bubble life, when it reaches the surface and bursts, releasing its gas content to the atmosphere. The disturbance of the free water surface may be responsible for some gas exchange with the atmosphere. It is believed (7) that the first stage of the life of a bubble in the column is characterized by higher transfer rates than in the intermediate steady phase where the transfer rate remains essentially constant irrespective of column height.

Bubble Formation

In the process of bubble formation there are two major factors:

1. Hydrostatic pressure, H, which is dependent on the height and density of the solution in the column above the tip of the capillary.

2. The back pressure, p, due to surface tension, which is a function of the radius of the bubble and the surface tension at the gas-liquid interface. This has been evaluated as $p = 2\sigma/r$, where σ = surface tension in dynes per cm and r = radius of the bubble in cm.

Bubbling begins when the total gas pressure, P, exceeds the sum of H and p. Thus, at constant hydrostatic head and on applying a constant gas pressure, P, enough to start a steady state of bubbling, the size of the bubble formed will decrease with decreasing values of liquid surface tension. Concomitant with this, at constant gas flow, is a decrease in the time of bubble formation. Pure water has the highest surface tension, the time for bubble formation is highest, and the biggest bubbles are formed (Figure 5). But as the concentration of the surface active agent increases from zero, the interfacial tension at the gas-liquid interface drops rapidly and smaller bubbles are formed at higher rates. The partial pressure inside these bubbles depends on: the mechanical and physical properties of the bubble wall, the hydrostatic head on the bubble which decreases as the bubble rises in the aeration column, and the change of the gas content inside the bubble during its ascent to the surface.

The effect of surface active agents on the process of bubble formation may be explained by considering surface viscosity of the gas-liquid interface. In pure water, having the highest surface tension and the lowest surface viscosity, the interface is highly elastic. This elastic interface above the tip of the capillary will not yield easily to the gas pressure, thus large bubbles are formed, the size depending to a great extent on the interface yield value. On the addition of surface active agent, adsorption at the interface rapidly lowers the surface tension values. This is accompanied by a proportional increase of the surface viscosity and

lower yield values of the interfacial adsorbed film. In this case, the force exerted by the gas on the interface above the tip of the capillary will cause shearing of the film. As a result, gas will break through forming small bubbles the size of which depends greatly on the degree of shearing. The photographic studies on the mechanism of bubble formation confirm this reasoning.

Bubble Dynamics

Once the bubble is detached from the surface of the capillary it passes through various stages of circulation (Figure 8) until it acquires a definite shape and a steady state of ascent. At higher Aerosol O.T. concentrations, the bubbles lose their flexibility and travel as if they are small rigid spheres. These bubbles cannot withstand the hydrodynamic effect of the circulating solution. Strains in the bubble wall cause it to shear and bubble splitting takes place. With increasing concentrations, more molecular packing takes place at the interfacial adsorbed layer on the bubble wall. This, plus the effect of surface orientation, produces increasing coherence in the adsorbed monolayer. The higher the concentration, the lower the rate of mass transfer across the adsorbed film and the more compact will be the interfacial film. The Newtonian character may still be maintained, however, and the bubble wall might acquire a plastic character with a true yield value. During this, the surface adsorbed film might pass through a process of phase transfer approaching the character of a rigid film. Accordingly, the bubble wall passes from an elastic contractile, to plastic, and upon increasing the concentration, it approaches a rigid character, the extent of which depends on the degree of shear (29). The mathematical treatment explaining the phase transfer in surface film monolayers is discussed by Temperley (30).

The stability of the bubble in its ascent in a column depends on the elasticity of the bubble wall to meet mechanical shocks. In the absence of surface active agents or at very low concentrations, the bubble will act as a highly flexible elastic membrane that can respond to mechanical shocks and maintain slight differences in tension throughout its surface representing a differential in stress. At high concentrations of surface active agents, the bubble wall is rather rigid and fragile and the adsorbed surface film may have the character of a solid film. Within the surface film, the higher viscosity will lead to internal shear that will oppose differences in tension in its different parts. Such a bubble wall cannot stand mechanical shocks, and excessive shearing caused by varying stress leads to bubble splitting. Figure 8 shows that at 80 mg/l of Aerosol O.T. the splitting of the bubbles takes place soon after they are detached.

The addition of 20 ml of paraffin oil in the aeration column apparently gave more stability to the Aerosol O.T. adsorbed film at the gas-liquid interface. Actually, the presence of the paraffin oil interfered with the phenomenon of phase transfer in the bubble wall. In the meantime it acted as a condensing agent for the Aerosol O.T. adsorbed film. Such film gave the highest resistance to gas transfer, 75-per cent reduction of the over-all oxygen transfer coefficient as shown in Figure 3. It is also important to note that in presence of paraffin oil impurities, there was no recovery for the K_La value, it remained at a more or less constant value, yielding a curve similar to that for K_L, which measures transfer per unit of surface area.

Conclusions

Aeration as considered in this study is a surface phenomenon. The gas-liquid interface is the dominating factor in the rate of mass transfer. Impurities, particularly if surface active

in nature, lower the rate of mass transfer in this two-phase system. Thus, surface active agents usually exert great influence on the rate of transfer. The surface active agent chosen for this study, Aerosol O.T., is anionic in nature, with well-established structural formula and physicochemical characteristics, and similar to ABS in its effects.

The effect of the surface active agent at the interface in this particular system is summarized as follows:

1. On the addition of small amounts of the Aerosol O.T., the rate of oxygen transfer was reduced to a minimum value. The concentration at which this minimum occurred corresponded to the critical micelle concentration (CMC) of the Aerosol O.T. in solution. Further addition of the surface active agent did not affect the rate of gas transfer per unit area of interface, K_L.

2. Increase of the Aerosol O.T. concentration beyond the CMC tends to create more exposed interfacial area. This is accompanied by a gradual increase in the over-all rate of transfer K_La.

The rate of K_La recovery is influenced by the total exposed surface area and surface impurities. With increasing surface active agent concentration, there is a gradual increase in the total exposed surface area. This is caused primarily by the formation of smaller bubbles for the same rate of oxygen flow as well as bubble splitting during the ascent in the aeration column.

Bubble formation and bubble volume depend on the gas-liquid interfacial conditions. Reducing the surface tension causes smaller bubbles to form and increasing the surface viscosity causes bubble splitting. Surface active substances affect the surface tension and surface viscosity which in turn affect the interface characteristics. The addition of 20 ml of paraffin oil in the aeration column prevented almost completely the recovery of the K_La value.

With a diffuser instead of a single capillary, there is a gradual increase of K_La with increasing concentration of the surface active agent. Despite the reduction in the absolute transfer coefficient, K_L, the over-all oxygen transfer coefficient K_La was found to be greater than that for clear water and it increased with increasing Aerosol O.T. concentrations. This is caused by the great increase in the exposed surface area afforded by the very small bubbles from the diffuser, which minimizes the effect of the interfacial adsorbed surface active film.

Acknowledgments

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Industrial Wastes

TREATMENT OF ORGANIC WASTES IN AERATED LAGOONS *

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Lagooning affords an economical method of waste treatment where large land areas are available, and it is particularly appropriate for seasonally operated plants. Principal objections to lagooning are odors in the surrounding area and the large capacity requirements dictated by the comparatively low rate of oxidation under anaerobic conditions. These objections may be overcome by the installation of diffused or mechanical aeration systems in the lagoon. It is the purpose of this paper to discuss the application of the principles of aeration and oxidation to the design of lagoons.

Lagoons are employed in the treatment of both domestic and industrial wastes. Typical installations and operating results on domestic sewage have been studied and described (1) in which emphasis has been placed on shallow depths utilizing algae as an oxygen source. Application of lagooning to cannery wastes has been extensive, frequently employing sodium nitrate as a source of oxygen. In the treatment of paper mill wastes, oxygen is introduced by means of recirculation of the lagoon contents by pumping (2). Introduction of oxygen by bubble aeration, which resulted in a high rate of circulation of reservoir contents, has been described (3). This concept has been used in the lagoon treatment of paper mill wastes (4).

* Presented at the Spring Meeting of the New York Sewage and Industrial Wastes Assn.; Liberty, N. Y.; May 26–27, 1958.

Theory

The oxidation of organic matter under aerobic conditions in lagoons is analogous to the natural purification process occurring in rivers. Aerobic conditions are maintained provided the rate of oxidation does not exceed the rate of reaeration. When the concentration of organic matter is of such a magnitude that the oxidation rate exceeds the reaeration rate, anaerobic conditions may result. To maintain an aerobic environment in lagoons, it is usually necessary to supply additional oxygen by artificial means.

Under limiting conditions, the rate of biological oxidation of waste matter can be expressed by a first-order reaction as follows:

$$\frac{dL}{dt} = kL \quad \ldots\ldots\ldots\ldots (1)$$

This equation states that the rate of oxidation is proportional to the concentration of organic matter present at any time. The coefficient k is a measure of this rate and is a function of temperature. The constant k is expressed to the base e; however, values are commonly reported to the base 10 and the coefficient identified as K. Some doubt about the fundamental validity of this equation has been expressed by numerous investigators, but it remains a convenient and quantitative measure of biological oxidation.

In considering the rates of various

TABLE I.—Comparison of Biological Systems

System	Solids	Turbulence	Oxygen Concentration	Range K
Unaerated lagoon	Low	None	None	0.02–0.10
BOD bottle	Low	None	High	0.10–0.30
Natural rivers	Low to intermediate	Intermediate to high	Intermediate	0.10–1.00
Activated sludge	High	High	Intermediate	2.0 –10.0

biological oxidative systems, other factors must be taken into account, such as the concentration of active solids in suspension, intensity of fluid turbulence, and the level of dissolved oxygen. In Table I the effect of these factors on the coefficient K is indicated.

Since the solids and turbulence levels in an aerated lagoon are of the same order as those in natural streams, it may be anticipated that the magnitude of the removal coefficient would likewise be of the same order.

In a large aerated lagoon there is usually sufficient turbulence to create a condition of uniform concentration. It follows, therefore, that the concentration of BOD in the effluent is substantially equal to the concentration of BOD in the lagoon itself. To evaluate basin efficiency under these conditions and the conditions imposed by Equation 1, a material balance of the BOD may be developed as follows:

BOD applied − BOD discharged = BOD removed

$$l_a Q - l_e Q = (kl_e)V \quad \ldots\ldots (2)$$

in which

l_a = concentration of BOD in influent,
l_e = concentration of BOD in effluent,
Q = rate of flow, and
V = volume of lagoon.

Equation 2 may also be expressed in the following manner:

$$\frac{l_e}{l_a} = \frac{1}{1 + kt} \quad \ldots\ldots (2a)$$

or in terms of per cent removal efficiency

$$E = 100\,\frac{kt}{1 + kt} \quad \ldots\ldots (3)$$

In cases where the rate of oxidation is independent of concentration and is linear with time, the removal efficiency may be expressed:

$$E = \frac{100}{l_a}\,k_o t \quad \ldots\ldots (4)$$

in which

k_o = rate of BOD removal per unit time, and
t = detention time.

The rate of removal of BOD for any particular waste may be determined experimentally in the laboratory under conditions simulating solids and turbulence levels which may be expected in a large-scale installation. Employing the reaction rates developed from laboratory data, the detention time and volume of the lagoon may be determined for a given removal in accordance with either Equation 3 or 4.

A general relation between the removal of organic matter (BOD) and oxygen utilization for any bio-oxidation process has been expressed as follows (5):

$$R_r = aL_r + bS \quad \ldots\ldots (5)$$

in which

R_r = oxygen utilized, lb/day;
a = oxygen utilization factor based on fraction of organic matter oxidized in 5 days;
L_r = organic matter removed, lb/day (5-day basis);
b = fraction of sludge solids oxidized, per day; and
S = sludge solids in suspension, lb.

In an aerated lagoon, the sludge solids in suspension are negligible and the term bS_r may be eliminated from Equation 5. Hence,

$$R_r = aL_r \ldots\ldots\ldots\ldots (6)$$

The constant a may be evaluated from laboratory or field data. This constant is employed to determine the total oxygen requirement for any BOD removal level in the lagoon.

Aeration

In a lagoon, oxygen is transferred from the atmosphere to the liquid body through the surface. Additional oxygen, if required, may be transferred by mechanical or diffused aeration systems. The total quantity of oxygen transferred to the lagoon may be defined by the following expression:

$$N = N_s + N_D \ldots\ldots\ldots (7)$$

or

$$N = K_L A (C_s - C) + K_L' A' (C_s' - C) \ldots\ldots (7a)$$

in which

N = total quantity of oxygen transferred per unit time,
N_s = quantity of oxygen transferred by surface aeration per unit time,
N_D = quantity of oxygen transferred by mechanical or diffused aeration per unit time,
C = average dissolved oxygen concentration,
K_L = liquid-film coefficient for surface aeration,
A = surface area of lagoon,
C_s = saturation value of oxygen at atmospheric pressure,
K_L' = liquid-film coefficient for mechanical or diffused aeration,
A' = total surface area of the bubbles, and
C_s' = saturation value of oxygen for bubbles at the average depth of the lagoon.

Because there is no practicable way to determine the bubble interfacial area, Equation 7a is usually expressed:

$$N = K_L A (C_s - C) + K_L' a V_T (C_s' - C) \ldots\ldots (8)$$

where $K_L'a$ is the over-all liquid-film coefficient and V_T the volume of the lagoon.

From the data currently available, the concentration in the liquid body, C, should be maintained above approximately 1.0 mg/l. Below this level, the uptake rate has been found to be oxygen-concentration dependent.

A theoretical development defining the liquid-film coefficient, has been presented (6) whereby:

$$K_L = (D_L r)^{\frac{1}{2}} \ldots\ldots\ldots (9)$$

where D_L is the diffusivity of oxygen and r the rate of surface removal.

A theoretical approach has been developed (7) which relates the rate of surface renewal to parameters of fluid turbulence. For the case of non-isotropic turbulence, these parameters may be expressed in terms of the velocity gradient at the surface. It is this condition which is representative of surface aeration in a lagoon. The liquid-film coefficient may be defined as follows:

$$K_L = \left(D_L \frac{dU}{dy} \right)^{\frac{1}{2}} \ldots\ldots (10)$$

in which $\frac{dU}{dy}$ is the velocity gradient at the surface.

The velocity gradient is induced by the fluid motion in the surface layers of the lagoon. This motion may be generated by diffused- or mechanical-aeration equipment. Depending on the type and arrangement of the equipment, the supply of air could be a point or line source. In either case, the fluid motion and its associated turbulence characteristics decay exponentially with distance from the source. Figure 1 diagrammatically illustrates these cases. Typical vertical velocity

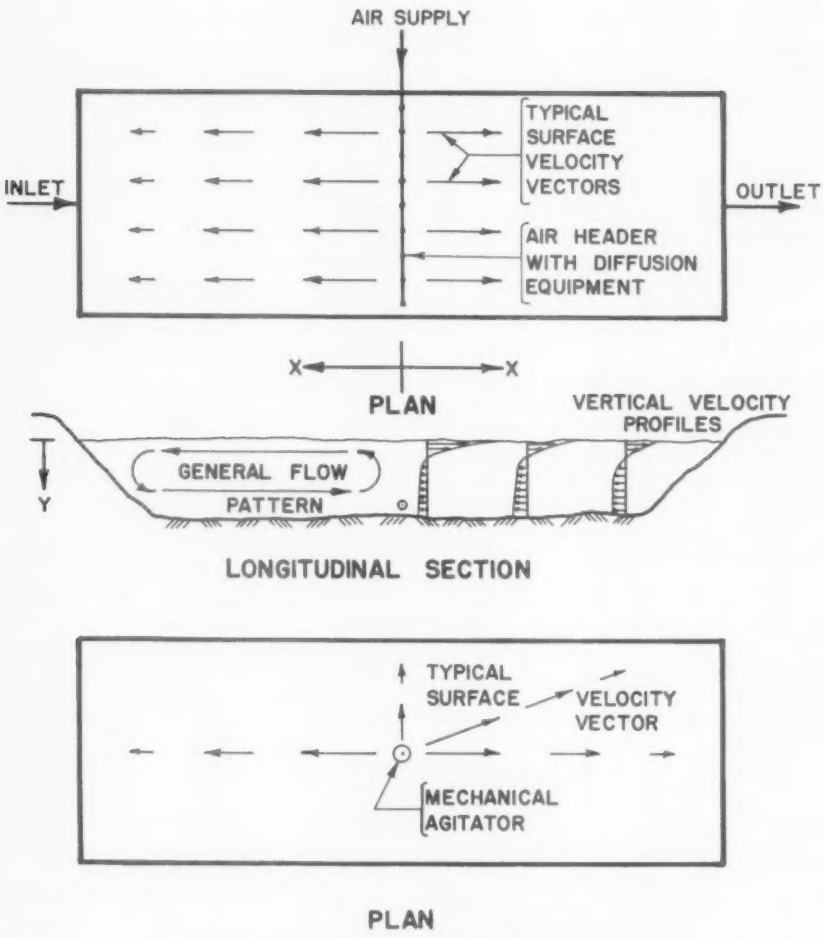

FIGURE 1.—Typical plan and profile of aerated lagoon.

profiles are also shown in this figure. In employing Equation 10 for design purposes, a mean value of the velocity gradient must be used. The high-velocity streamlines are found only in the upper layers moving away from the source, while the lower layer contains the low-magnitude values in the reverse direction. It is pertinent to note that the surface area directly above the air source is subject to violent agitation and surface renewal is properly characterized by isotropic turbulence, but the oxygen transferred in this fashion is not accounted for in Equation 10. This zone exists only in the immediate vicinity of the source, however, and is not representative of the entire lagoon surface.

From studies on the diffusion of submerged jets (8), it is reasonable to assume that the velocity should decay in some exponential fashion with depth and distance. Under certain conditions, the film coefficient K_L and the interfacial area in this localized zone may be significantly higher than those some distance from the source. The

decay of velocity with depth may be approximated by the relationship:

$$U = U_o e^{-fy} \quad \ldots\ldots\ldots (11)$$

in which

U = velocity at depth y,
U_o = velocity at the surface ($y = 0$),
f = constant, and
y = depth.

To obtain the velocity gradient at the surface, Equation 11 is differentiated with respect to y as follows:

$$\frac{dU}{dy} = -fU \quad \ldots\ldots\ldots (12)$$

and at the surface $U = U_o$ and Equation 12 becomes

$$\frac{dU}{dy} = -fU_o \quad \ldots\ldots\ldots (13)$$

Equation 13 may be employed to compute the velocity gradient at any distance, x, from the aeration source. To calculate the total surface aeration, an average value of the velocity gradient over the entire lagoon surface is required. To this end, an emperical relationship has been developed relating the velocity gradient with distance from a line source:

$$\frac{dU}{dy} = \frac{\phi}{x} \quad \ldots\ldots\ldots (14)$$

in which ϕ is a constant.

An average value of the velocity gradient may be obtained by integrating this function over the distance $x_1 \rightarrow x_2$. The distance x_1 is selected as one foot from the source because of the indeterminate nature of the velocity gradient above the source. The distance x_2 is one-half the spacing of the air headers. The average velocity gradient is therefore equal to:

$$\overline{\frac{dU}{dy}} = \frac{\int_1^{x_2} \frac{\phi}{x}\, dx}{x_2 - x_1} = \frac{\phi[\ln x]_1^{x_2}}{x_2 - 1} \quad \ldots (15)$$

The average velocity gradient computed from Equation 15 is substituted in Equation 10 to determine the average coefficient of surface reaeration. Measured velocity gradients have ranged from 0.1 to 4.0 per second. These are of the same order as those found in natural streams.

Oxygen absorption by diffused aeration is related to the submergence depth of the diffusion device, the air-flow rate, and the characteristics of the waste being treated. A general relationship for any diffusion device has been developed (9):

$$K_L'a = \frac{cG_s^n H^m}{V_o} \quad \ldots\ldots\ldots (16)$$

where

$K_L'a$ = the over-all oxygen transfer coefficient for the air diffusion device;
V_o = the lagoon volume per air-diffusion unit, cu ft;
G_s = air-flow per diffusion unit, cfm @ standard conditions;
H = average depth of lagoon, ft; and
c, m, n = constants characteristic of the aeration device.

For most diffusion devices m will vary from 0.6 to 0.8 and n from 0.5 to 1.5.

The over-all oxygen balance in the lagoon can be expressed by the equation:

$$\frac{dc}{dt} = K_z(C_s - C) - r_r \quad \ldots (17)$$

When steady-state conditions exist, Equation 17 becomes:

$$K_z(C_s - C) = r_r \quad \ldots\ldots (18)$$

in which K_z is the over-all transfer coefficient and r_r the oxygen-uptake rate.

Equation 18 may be employed to determine the over-all coefficient K_z which measures the combined effect of surface and diffused aeration. The term r_r is measured in the field or laboratory and is related to BOD removal.

Temperature Effects

Both the rate of biological oxidation and the rate of oxygen transfer are temperature dependent. The relationship commonly employed is:

$$K_T = K_{20} \cdot \theta^{(T-20)} \quad \ldots\ldots (19)$$

where K_T and K_{20} are the reaction rates at temperature T (°C) and 20°C, respectively, and θ is a constant. The value of θ for biological oxidation has been reported to vary from 1.035 to 1.065. In this study, a temperature coefficient of 1.035 was obtained from laboratory and field investigations. Further study should be made to confirm the temperature coefficients for the oxidation of various wastes. The value of θ for oxygen transfer has been reported to vary from 1.016 to 1.047.

Sludge Accumulation

A portion of the BOD removed in the lagoon results in the synthesis of microbial sludge. A significant portion of these solids may settle to the bottom, and, in addition, solids present in the influent may also settle. Sludge accumulation has been found to vary from 0.1 to 0.2 lb/lb BOD removed. Further studies are being conducted to evaluate this factor.

Laboratory Investigations

Laboratory investigations were carried out under aerobic conditions similar to those which may occur in the actual operation of a lagoon to determine BOD removal characteristics. Aerobic conditions were maintained by the natural process of atmospheric reaeration induced by a vertically oscillating grid suspended in the waste sample. The grid did not break the water surface. Additional batch studies were conducted in which aerobic conditions were maintained by bubble aeration. Cannery and paper mill wastes were studied; a typical reaction rate curve is shown in Figure 2.

To compare the efficiency of a continuous process with a batch process and check the validity of Equation 3, a 29-gal drum was fed continuously and operated in parallel with the batch operation. Detention periods of one to five days were employed. The rates of oxidation of cannery wastes were measured under these conditions at constant temperature. A synthetic waste was prepared using dietetic tomato juice and corn puree diluted to concentrations typical of these wastes. Seed material from a previous run was added to each batch sample; the amount was 10 per cent by volume. Daily determinations were made of DO, BOD, COD, SS, pH, and oxygen-uptake

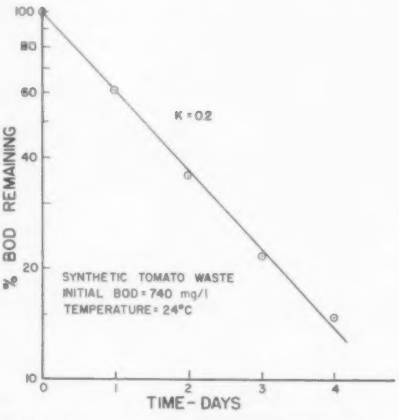

FIGURE 2.—Typical BOD removal curve for cannery waste.

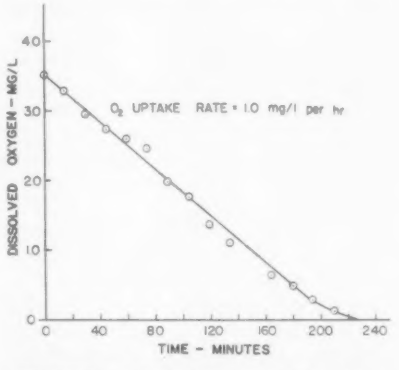

FIGURE 3.—Typical oxygen-uptake rate.

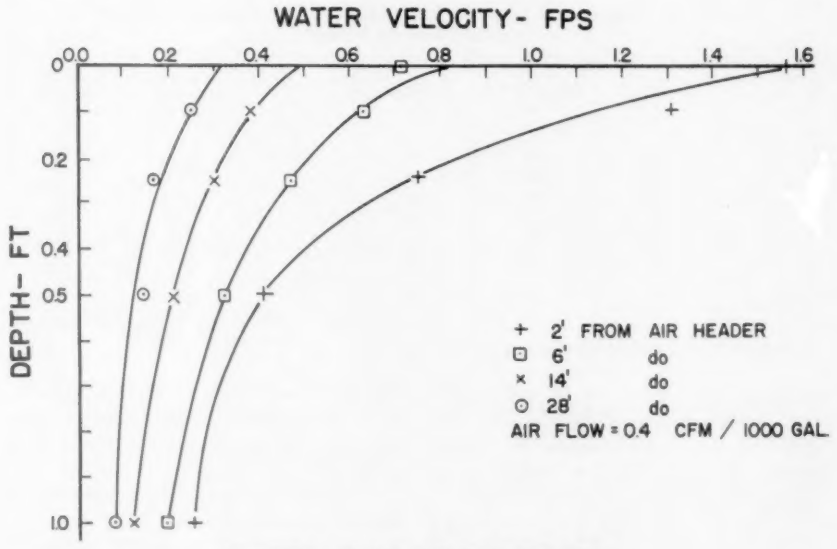

FIGURE 4.—Typical vertical velocity profiles.

rate. The oxygen-uptake rate was determined by measuring the dissolved oxygen polarographically in a sealed, agitated sample at selected time intervals. Depending on the oxygen-uptake rate and the waste characteristics, the time intervals varied from 5 to 15 min. Typical data are shown in Figure 3.

Field Investigations

Field studies were conducted in a cannery-waste lagoon 240 ft long, 120 ft wide, and 5½ ft deep. Two air headers, 120 ft apart, were placed across the width of the lagoon. Air spargers were spaced at 5-ft centers on each header. The waste flow was distributed at one end of the lagoon and the treated waste discharged over a weir at the other end. Additional studies were conducted on an experimental lagoon treating pulp and paper mill wastes (4). This lagoon was 63 ft long and 43 ft wide with an average operating depth of 9.5 ft. Aeration was accomplished by three sparger units placed on 9-ft centers on an air header across the center of the lagoon.

Velocity gradients were estimated from velocity measurements made with a standard current meter at various depths and distances from the air headers. A typical set of velocity profiles is shown in Figure 4. Only the velocities in the upper six inches are significant in evaluating the velocity gradient at the surface.

Daily measurements of BOD, COD, and SS were made on influent and effluent samples. Dissolved oxygen, temperature, and oxygen-uptake rates also were measured.

Results

The results of the laboratory investigations on continuous treatment of cannery wastes are summarized in Table II. These studies were conducted at temperatures ranging from 24 to 26°C. Pilot-plant data for a paper mill waste are shown in Table III. The operating temperature for the paper mill waste oxidation varied between 15 and 36°C. The per cent BOD reductions were adjusted to 25°C in the case of the paper mill waste using a

TABLE II.—Summary of Data from a Continuously Operated Laboratory Unit Treating Cannery Wastes

Run	Feed BOD (mg/l)	Effluent BOD (mg/l)	Temp. (°C)	Detention (days)	Rate* Constant, K (per day)	BOD Removal (%)
1	580	170	25	5	0.21	71
2	550	180	26	4	0.30	70
3	550	270	24	4	0.24	61
4	890	370	26	3	0.22	60
5	840	340	26	2	0.17	59
6	605	345	24	1	0.17	43
Mean	—	—	25	—	0.22	—

* Rate constant from simultaneous batch-type unit.

temperature coefficient, θ, of 1.035. The oxidation-rate constant was determined for both wastes by batch laboratory studies, and a typical curve for the cannery wastes studied is shown in Figure 2. The rate for both wastes varied from 0.17 to 0.30 per day at an average temperature of 25°C. The numerical values of the rate constant for most of the data ranged from 0.18 to 0.26. Based on these findings, theoretical reductions were computed from Equation 3. A comparison of the theoretical and observed data is shown in Figure 5. The relationship between BOD removed per unit of lagoon volume and oxygen utilization in the lagoon is shown in Figure 6.

The surface aeration coefficient was determined from velocity profiles measured in the field at various air flows. The velocity data shown in Figure 4 are replotted in Figure 7. From the intercept and slope of these relationships, the velocity gradient is computed in accordance with Equation 13.

The velocity gradients computed in this manner for various distances from the air source are shown in Figure 8. The constant ϕ in Equation 14 is evaluated from this plot. The mean velocity gradient is then determined by means of Equation 15 for the appropriate header spacing.

To evaluate the coefficient of diffused aeration in a particular waste it is necessary to eliminate the surface aeration effects as previously described. This evaluation can be made by employing a non-steady state aeration procedure in a tank in which the air diffusion unit is placed in the center. The

TABLE III.—Summary of Data from a Pilot Plant Treating Paper Wastes

Run	Feed BOD (mg/l)	Effluent BOD (mg/l)	Temp. (°C)	Detention (days)	BOD Removal*† @ 25°C (%)
1	210	140	27	1	32
2	250	145	23	2	43
3	250	130	20	2	51
4	200	55	31	3	68
5	250	90	15	4	70
6	290	120	20	4	62
7	260	100	25	4	62
8	210	40	30	4	76

* K from batch oxidation, 0.18 to 0.26 @ 25°C.

† Corrected by $K_T = K_{20}\ 1.035^{(T-20)}$.

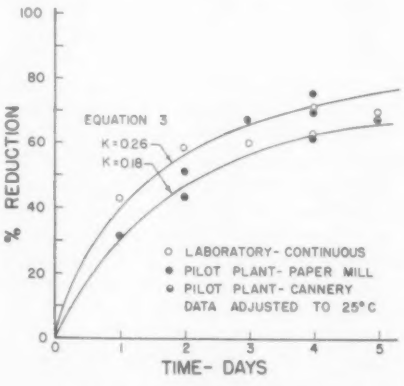

FIGURE 5.—BOD reduction for continuous operation.

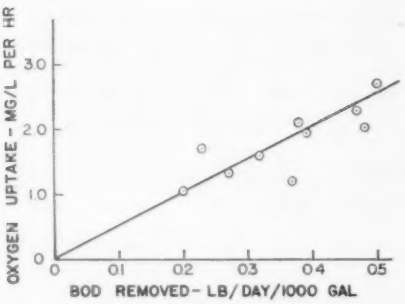

FIGURE 6.—Oxygen uptake related to BOD removed.

surface area of the tank is very small compared with that of the lagoon. A ratio of 50:1 was used in this study. The dissolved oxygen initially present is removed with cobalt-catalyzed sodium sulfite. The rate of absorption of oxygen at various air rates and various depths is determined by the change in dissolved oxygen content after oxidation of the sulfite is complete.

The over-all transfer coefficient, K_s, for the lagoon was computed by means of Equation 18. The total amount of oxygen transferred, determined by this means, compared favorably with the calculated amounts determined by substituting, consecutively, the computed coefficients for surface and diffused aeration in Equation 8 (Table IV). The values computed by means of Equation 18 were consistently greater than the values computed from Equation 8 which may be attributed to the surface aeration in the immediate vicinity of the air source.

A study was conducted to evaluate the effect of mixing in the experimental

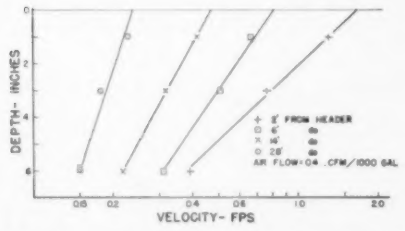

FIGURE 7.—Variation of velocity with depth.

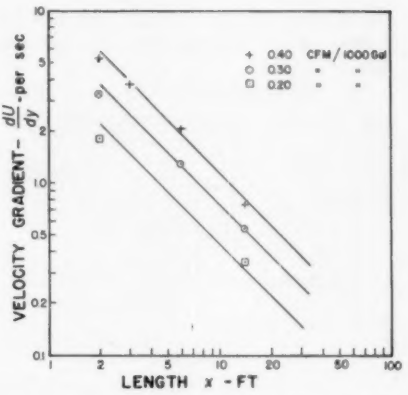

FIGURE 8.—Decay of velocity gradient with distance.

lagoon. Two procedures were employed: dye dispersion and simultaneous determination of oxygen-uptake rate and COD at various locations in the lagoon. The results of these tests indicated that there was sufficient turbulent mixing to establish substantially homogeneous conditions throughout the lagoon. In the 240-ft lagoon, with an air flow of 0.5 cfm/1,000 gal,

TABLE IV.—Comparison of Oxygen Transfer Values

Air Flow (cfm/1,000 gal)	Oxygen Transferred* (lb/hr)			
	Calculated from			
	Equation 8			Equation 18
	Surface	Diffused	Total	Total
0.40	0.43	0.90	1.33	1.46
0.50	0.51	1.22	1.73	1.78

* Based on 1.0 mg/l DO in the lagoon at 25°C.

TABLE V.—COD Values

Location	COD (mg/l)
Infl.	1,650
60 ft from infl.	1,110
120 ft from infl.	1,040
180 ft from infl.	1,080
Effl.	1,090

the measured COD values at various points in the lagoon were as shown in Table V.

The BOD removals obtained in the experimental lagoon ranged from 50 to 80 per cent for detention periods of from 2 to 5 days, with temperatures ranging from 15 to 30°C. These were of the same order of magnitude as those obtained in the laboratory.

Design Example

1. *Data*

   a. Flow = 1 mgd
   b. BOD raw = 250 mg/l
   c. Efficiency = 70 per cent @ 25°C

2. *Volume of basin*

   a. From laboratory data, the oxidation rate $K = 0.23$ @ 25°C
   b. Detention time from a rearrangement of Equation 3:

$$t = \frac{E}{K(100 - E)}$$

$$t = \frac{70}{0.23 \times 2.3(100 - 70)}$$

$$t = 4.5 \text{ days}$$

   c. Volume:

$V = 4.5$ days $\times$ 1.0 mgd
$V = 4.5$ mil gal = 600,000 cu ft
Use 2 units each @ 300,000 cu ft

   d. Dimensions:

Assume
Depth = 10 ft
Area = 30,000 sq ft
Length = 300 ft
Width = 100 ft

3. *Design of aeration system*

   a. From laboratory data, the BOD–O_2 relationship was found to be (Equation 6):

$R_r = aL_r$
1.1 lb/day O_2 is required to satisfy 1.0 lb/day 5-day BOD thus
O_2 needed $= 1.1 \times 0.70 \times 250 \times 8.34 \times 1.0$
$= 1{,}600$ lb/day = 67 lb/hr

   b. A diffusion device was selected with an efficient operating range of from 5 to 30 cfm per unit. For the first trial design, the following parameters were assumed:

Air flow per unit = $G_s = 15$ cfm
Air header spacing = 60 ft
No. of headers per basin = 5
DO concentration = $C = 1.0$ mg/l
DO saturations @ 25°C—
$C_s = 8.4$ mg/l
$C_s' = 9.0$ mg/l @ average depth

c. The total oxygen transfer is expressed by Equation 8 which may be re-expressed in terms of the number of diffusion units, n:

$$N = K_L A (C_s - C) + nK_L' a V_o (C_s' - C)$$

From this equation, the number of diffusion units may be determined.

d. Surface aeration:

Assume air flow = 0.4 cfm/1,000 gal, then for this air flow, using the plot of Figure 8, Equation 14 becomes

$$\frac{dU}{dy} = \frac{11.5}{x}$$

The mean velocity gradient is (Equation 15)

$$\frac{dU}{dy} = \frac{11.5[\ln x]_1^{30}}{30 - 1} = \frac{11.5 \times 3.4}{29} = 1.35/\text{sec} = 4{,}900/\text{hr}$$

(The upper limit of 30 ft is $\frac{1}{2}$ the header spacing.)

The surface aeration coefficient may be now computed from Equation 10

$$K_L = \left[D_L \frac{dU}{dy} \right]^{\frac{1}{2}}$$

for 25°C, $D_L = 0.93 \times 10^{-4}$ sq ft/hr, and so

$$K_L = \sqrt{0.93 \times 10^{-4} \times 0.49 \times 10^{4}}$$

$$= 0.67 \text{ ft/hr}$$

Because the surface area = 60,000 sq ft

$$N_s = K_L A [C_s - C]$$
$$= 0.67 \times 60{,}000\ [8.4 - 1.0] \times 62.4 \times 10^{-6}$$
$$= 19 \text{ lb/hr}$$

e. Diffused aeration:

The required oxygen transfer from diffused aeration equals

$$N_D = N - N_s$$
$$= 67 - 19 = 48 \text{ lb/hr}$$

The number of diffusion units may be determined from

$$N_D = nK_L' a V_o (C_s' - C)$$

For $G_s = 15$ cfm and lagoon 10 ft deep $K_L' a V_o = 750$ cu ft/hr

$$\therefore \quad n = \frac{48}{750 \times (9.0 - 1.0) \times 62.4 \times 10^{-6}} = 129$$

The number of diffusion units per header $= \frac{129}{5 \times 2} = 12.9$; use 13 and space at 100/13 or 7.7 ft

Back check on assumed air flow of 0.4 cfm/1,000 gal:

$$\text{Air flow/1,000 gal} = \frac{129 \times 15}{4{,}500} = 0.43 \text{ cfm}$$

In a similar manner the design for other depths and conditions may be calculated and the most economical selection made.

Summary

The rate of BOD removal was determined in a laboratory batch treatment device which simulates lagoon conditions. The rate of oxidation determined from the laboratory data can be used to predict the efficiency of continuous lagoon systems from Equation 3 or 4.

The rates of oxygen uptake and solids accumulation are determined concurrently with the BOD removal study in the laboratory. From these data, a relationship between oxygen utilization and BOD removal can be developed. The total oxygen requirements may be determined for any BOD removal level. These requirements are supplied by oxygen transferred from surface and diffused aeration. The coefficient of surface aeration is related to the average velocity gradient at the water surface of the lagoon. The coefficient of diffused aeration is related to the nature of the diffusion device, the air flow, and the lagoon depth. Field studies were employed to determine surface and diffused aeration.

Temperature influences the rate of BOD removal and oxygen utilization. The rates of surface and diffused aeration are also related to temperature.

Conclusions

1. Aerated lagoons afford an economical method of waste treatment where land is available.

2. Laboratory and field procedures have been presented which may be employed for the design of aerated lagoons.

3. BOD removals of up to 80 per cent have been obtained on cannery and paper mill wastes.

4. Additional laboratory and field work are necessary to confirm the tentative conclusions suggested by this work.

Acknowledgment

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DISCUSSION

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Professors O'Connor and Eckenfelder are to be congratulated for pointing the way to a better basis of design for aerated-lagoon treatment units. Theirs is the first published paper in which the equations for surface reaeration developed by O'Connor and Dobbins (7) have been used in the design of aerated waste treatment units.

Space limitations undoubtedly account for the omission of detailed field data in their report, making it impossible to compare results with theoretical design considerations. Some of the questions raised by the present paper might have been answered had these data been published. There are, however, some factors in evaluating the oxygen requirements of a lagoon which seem significant to this writer, but they were either not considered or not taken into account in the report. Some are of minor importance, but because all require a plus correction, i.e., an increased oxygen requirement, their cumulative effect is large. Discussion of these factors herein is not necessarily in order of importance.

Benthal Oxygen Demand

The solids which settle to the bottom of a lagoon continue to exert an oxygen demand by benthal oxidation. This process, which is somewhere between aerobic oxidation and anaerobic decomposition, might not appear important because its oxidation-rate constant is between 1/30 and 1/40 that of domestic sewage (10) (11) (12). However, since the sludge solids accumulate in the system, they continue to exert a deoxygenation effect over a year or longer and thus are important when considering aeration requirements.

Fair *et al.* (10) and Lardieri (11) studied benthal oxidation in the laboratory, and from their data it can be assumed that the benthal oxygen demand per year will be equal to 25 per cent of the volatile solids deposited.

For the design example used by the authors, the benthal oxygen demand can be estimated as follows:

1. *Data*

   *a**. Flow = 1 mgd
   *b**. BOD raw waste = 250 mg/l
   c. SS raw waste = 250 mg/l
   d. VSS = 80 per cent
   *e**. BOD effluent = 75 mg/l
   f. SS effluent = 75 mg/l
   *g**. Temperature = 25°C
   h. Sludge solids formed per lb of BOD added (80 per cent volatile) = 0.15 lb
   i. Oxygen demand of sludge VS = 0.25 lb O_2 per year per lb of VS

2. *Oxygen requirement*

   a. Volatile solids:

   $$\text{VSS (raw waste)} = (250 - 75) \times 8.34 \times 0.8 = 1{,}170 \text{ lb/day}$$

   $$\text{VSS (biological oxidation)} = 250 \times 8.34 \times 0.15 = 310 \text{ lb/day}$$

   Total VS = 1,480 lb/day

* Assumed by the authors.

b. Benthal oxygen demand:

$$O_2 \text{ needed} = \frac{1{,}480 \times 365 \times 0.25}{365} = 370 \text{ lb/day}$$

(after 1 years' operation) = 15.5 lb/hr

Oxygen Requirements for BOD

O'Connor and Eckenfelder have expressed their BOD oxygen requirements as:

$$R_r = aL_r \quad \ldots\ldots\ldots\ldots (6)$$

where

R_r = oxygen utilized, lb per day;
a = oxygen utilization factor based on fraction of organic matter oxidized in 5 days; and
L_r = organic matter removed lb per day (5-day basis).

They state that the value of a can be evaluated from laboratory or field data, and have used a value of 1.1 for this constant in their design example, but, unfortunately, the method for calculating this value was not given. If the first-order equation is used for the BOD curve, it can be demonstrated mathematically, assuming the K values for the influent and effluent are the same, that:

$$R_r = \frac{L_1}{l_a}(l_{a1} - l_{e2}) \quad \ldots\ldots (20)$$

where

R_r = (as above) oxygen utilized, lb/day;
L_1 = ultimate BOD of the raw waste, mg/l;
l_a = 5-day BOD of the raw waste, mg/l;
l_e = 5-day BOD of the effluent, mg/l; and
$\frac{L_1}{l_a} = a$.

The value of a is about 1.1 for values of K between 0.18 and 0.25. However, these K values hold for BOD observations up to 5 or 6 days and give low results for the true value of the ultimate BOD, L. When observations are made over the entire sewage BOD curve (and for many other wastes), K approaches a value of 0.1 and the ratio of the ultimate to the 5-day BOD, a, approaches a value of 1.45. The change in the value of the first-order curve constants with time is shown by Orford and Ingram (13), and the errors that can occur when the first-order equation is used with short-term BOD observations are thoroughly discussed.

There are many wastes which oxidize completely in 5 days or less, some as low as 1 day. These latter wastes usually contain easily oxidized soluble and colloidal material with little or no suspended organic matter. With these wastes, the a value, based on 5-day BOD results, would be low, approaching a value of 1.0. Thus, to select a value for the constant a, some information about the entire BOD curve for both raw waste and effluent should be available, at least for the raw waste.

O'Connor and Eckenfelder have shown that there is very good agreement between calculated oxygen transfer and observed oxygen transfer. Thus, the low value of 1.1 obtained for a must be substantially correct for their work. It should be expected that in a lagoon, where the contents are thoroughly mixed, the amount of oxidation is less than will occur in a BOD test for the same average detention period. Since biological oxidation occurs at a diminishing rate, the portion of the waste whose detention time is less than average will have a much higher oxygen-demand rate than the portion of the waste whose detention time is proportionally greater than the average. Thus partially treated waste continuously leaves the lagoon, lowering the computed oxygen requirements. Also,

a portion of the BOD will be removed by sedimentation and will not require immediate oxidation. This emphasizes the need for considering the benthal oxidation requirements of the bottom sludge.

The low oxidation in mixed lagoons can be illustrated by using the authors' results for Run 1, Table II. Here the oxygen requirement for BOD can be computed as:

O_2 required

$$= a \times (\text{mg/l BOD change}) = aR_r$$
$$= 1.1 \times (580 - 170)$$
$$= 451 \text{ mg/l}$$

Thus, with a lagoon detention period of 5 days and a temperature of 25°C, the computed oxygen requirement is less than the 5-day 20°C BOD of the waste by 129 mg/l or 22 per cent. An important point is that the value for a of 1.1 does not necessarily hold for all mixed lagoons or all types of wastes. For longer detention times the value will rise to a maximum of about 1.4 and for shorter detention times, it may be less than 1.0. When the wastes have mostly soluble BOD, the value of a should decrease; the reverse should be true when wastes have a high proportion of organic suspended material.

Surface Reaeration

In calculating surface reaeration, the authors have made their greatest contribution. At the time of their writing, there was not available to them reliable information concerning the effect of organic material on the rate of oxygen transfer. Recently, Rand (14) has shown that the presence of sewage reduces the surface reaeration rate cont stant K_L by as much as 40 per cenwhen the BOD is 250 mg/l and by 25 per cent when the BOD is 75 mg/l. Rand does not suggest that BOD or the presence of organic material per se is responsible, but rather that BOD is the best measure of the materials thamay be causing the interference (proteins, detergents, and other compounds with strong surface effects).

TABLE VI.—Calculated Values for Surface Reaeration for Design Example

Air Flow (cfm/1,000 gal)	ϕ	K_L* (ft/hr)	N_s† (lb/hr)
0.2	4.4	0.33	9
0.3	7.2	0.42	11.5
0.4	11.5	0.54	14.7
0.5	14	0.63	17.2
0.6	19	0.69	18.8
0.7	24	0.78	21.2
0.8	30	0.87	23.7

* Values for K_L reduced 20 per cent—dissolved oxygen at saturation, 25°C = 8.1 mg/l.
† Oxygen transferred by surface aeration.

If the assumption is made that the lagoon has a BOD of 75 mg/l, it can be estimated that the surface reaeration will be reduced 20 per cent. This takes into account the fact that the wastes in the lagoon are partially oxidized and the effect will probably be lessened.

The authors developed an empirical relationship $\frac{dU}{dy} = \frac{\phi}{x}$ (Equation 14) from experimental data plotted in Figure 8. The writer has extended these curves by assuming an apparent logarithmic spacing between the lines and has estimated the values for the constant ϕ, which is merely the intercept on the $x = 1$ ft axis. These values are given in Table VI and will be used in subsequent calculations.

Another factor affecting reaeration is the oxygen saturation value for a given temperature. It has long been known that the oxygen saturation values commonly used and given in "Standard Methods" (15) are too high. Truesdale *et al.* (16) in 1955 reported new oxygen saturation values. Their work was done with great care, using modern equipment and refined techniques, and can be considered more accurate than the work done about 50 years ago. Gameson and Robertson (17) simplified the equations which Truesdale developed and found that for pure water

the equation for oxygen saturation is

$$C_s = \frac{475}{33.5 + T}$$

and for sea water the equation is

$$C_s = \frac{475 - 2.65S}{33.5 + T}$$

where

C_s = oxygen saturation value, mg/l;
T = temperature, °C; and
S = salinity as NaCl in parts per thousand.

For the design example used, the oxygen saturation value at 25°C should be 8.1 mg/l instead of 8.4 mg/l. This is a small difference, but when the dissolved oxygen level is 1.0 mg/l, the calculated reaeration is 4.2 per cent too great, and when the dissolved oxygen is at 50 per cent saturation (a common figure), the calculated reaeration is 7.5 per cent too great.

The authors' surface reaeration values for the design problem have been recalculated using reduced values for the reaeration constant, K_L and the oxygen saturation constant, C_s. Both constants were reduced as previously described. These values are given in Table VI.

It is interesting that the estimated benthal oxidation rate of 15.5 lb/hr of O_2 is about the same as the median estimated surface reaeration rate of 17.2 lb/hr. The writer suggests that for design purposes it be assumed that the surface reaeration rate equals the benthal oxidation rate.

The error involved when benthal oxidation is assumed to equal surface reaeration will be less than the error obtained when benthal oxidation requirements are assumed to be zero. Both surface reaeration and benthal oxidation should be considered if the ratio of surface reaeration to diffused aeration is great or the ratio of settleable organic solids to BOD is high.

Diffused Aeration

The effects of organic matter on the reaeration-rate constants for bubble aeration have not been determined. However, the principles for diffusion through plane surfaces and through bubble surfaces are similar, and it can be expected that the rate constants for bubble aeration will also be reduced by the presence of organic matter.

The reduction in reaeration rate cannot be evaluated at this time, but it can be given some consideration by increasing the factor of safety on minimum oxygen levels. Instead of setting 1.0 mg/l of oxygen as the working oxygen level in a lagoon, the minimum might be raised to 1.5 or 2.0 mg/l. This has approximately the same effect on estimated air requirements as reducing the bubble reaeration rate constant by from 7 to 14 per cent.

Air Diffusion Requirements

By assuming that surface reaeration equals benthal oxidation, the calculations for diffused air requirements are greatly simplified. The oxygen requirement for BOD removal, R_r, from the authors' calculations is 67 lb of O_2 per hour. To compare O'Connor and Eckenfelder's air piping design, the same design assumptions are used where possible:

Air flow per unit = G_s = 15 cfm
Air header spacing = 60 ft
Number of headers per basin = 5
Length of each header = 100 ft
DO minimum concentration = C = 1.5 mg/l
DO saturations @ 25°—

C_s = 8.1 mg/l
C_s' = 8.7 mg/l @ average depth

The number of diffusion units may be determined from

$$N = nK_L'aV_o(C_s' - C)$$

and from the authors' paper

$$K'_La V_o = 750 \frac{\text{cu ft}}{\text{hr}}$$

$$\therefore \quad n, \text{ the number of diffusion units} = \frac{67}{750(8.7 - 1.5) \times 62.4 \times 10^{-6}} = 200$$

The number of diffusion units per header $= \frac{200}{5 \times 2} = 20$ and diffuser spacing per header $= \frac{100}{20} = 5.0$ ft.

The total diffused air requirement $= 15 \times 200 = 3{,}000$ cfm and the total air flow $= \frac{3{,}000}{4{,}500} = 0.67$ cfm/1,000 gal.

Conclusions

The writer's methods of calculation give a 54 per cent greater requirement for air than O'Connor and Eckenfelder's. Even so, O'Connor and Eckenfelder have shown good agreement between their calculated and observed oxygen transfer quantities. Obviously, this disagreement between the two design approaches must either be reconciled or one considered wrong.

A few possible explanations for these differences are suggested:

1. The field lagoons were new and a bottom sludge had not developed to a point where it had a significant benthal oxygen demand.
2. The laboratory pilot lagoons had no bottom sludge to give a benthal oxygen demand.
3. The authors' constants for reaeration, shown in Figure 8, were developed from laboratory and field data. Thus, such effects as the reduction in reaeration rate caused by the presence of organic material and the reduction attributed to lowered dissolved oxygen saturation values have already been cancelled out in the observations and calculations.
4. If the assumptions above are true, then O'Connor and Eckenfelder's close agreement between the calculated air requirements and the observed air requirements would be expected.

The writer has attempted to point out some of the problems which should be considered in the design for the air requirements of an aerated lagoon. Each design presents a different problem, and it does not follow that the design constants which apply to one lagoon will necessarily be appropriate for the design of another where such things as detention time, temperature, type of waste and its oxidizability are all different. If such factors can be thoroughly studied in the laboratory, as the authors suggested and were able to do, then a less conservative design is dictated. But extensive laboratory and pilot plant studies are not always possible, and the writer suggests that a more conservative approach be used. Although O'Connor and Eckenfelder have made an important step toward the improved design of aerated lagoons, this writer insists that no matter what design approach is taken, some consideration must be given to the oxygen demand of the bottom deposits.

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ALGAE AND METROPOLITAN WASTES SEMINAR

Announcement was made in the March JOURNAL of a Seminar on Algae and Metropolitan Wastes, April 27–29, at the Robert A. Taft Sanitary Engineering Center in Cincinnati. This meeting is being sponsored by the Water Supply and Water Pollution Research Staff of the Public Health Service.

For those who can make last-minute plans to attend, it appears to be an excellent opportunity to learn the latest about the prevention and control of objectionable blooms of algae resulting from enrichment by urban and other wastes. An outstanding group of speakers will be on hand to discuss the problems.

Further arrangements and information can be obtained from Dr. Alfred F. Bartsch, Assistant Chief, Water Supply and Water Pollution Research, Robert A. Taft Sanitary Engineering Center, 4676 Columbia Parkway, Cincinnati 26, Ohio.

THIS YEAR IT'S IN PHILADELPHIA

This year's big Federation meeting—the 33rd—is to be in Philadelphia, October 2–6. The Sheraton will be the headquarters hotel and meetings and exhibits will be in Convention Hall. Plan now for "the meeting of the year."

PHENOLIC WASTES FROM AIRCRAFT MAINTENANCE

By George W. Reid, Robert Daigh, and Robert L. Wortman

Professor of Civil Engineering and Sanitary Science, University of Oklahoma, Norman Okla.; Sanitary Engineer, Hq AMC, Wright-Patterson Air Force Base, Ohio; and Graduate Student, University of Oklahoma, Norman, Okla.

Maintenance of aircraft creates a unique problem in industrial waste, particularly phenol waste. For many years the Air Force has been concerned with the problems arising from the discharge of phenolic waste. The numerous maintenance installations throughout the states, and the variable effluent standards create the problems. Of those installations discharging to open streams the phenol standards range from 0.005 to 5.0 mg/l. At one installation the effluent standard is 50 mg/l. However, the discharge is to a sanitary district system where adequate dilution is available before treatment in a trickling filter plant.

Phenolic wastes at Air Force installations arise largely from the cleaning and processing of aircraft components and engine parts. The phenol is an expression of the cresylic content used in "Carbon Remover Compound." This product is purchased on the following specification: 25-per cent cresylic acid, 60-per cent orthodichlorobenzene, wetting agents, penetrants, and soaps and detergents.

In the engine repair shops, where the majority of carbon remover compound is used, phenol concentrations range from 200 to 400 mg/l in the rinse water. Depending on the dilution afforded in the industrial waste sewer system, the phenol concentration at the influent to the industrial waste treatment plant may vary from a few milligrams per 1,000 liters to several hundred milligrams per liter.

Two of the Air Force industrial waste treatment plants were constructed having specialized phenol treatment facilities. One plant has a closed system recirculating the "clean" phenol water back to the maintenance shop. In this plant, approximately 6,000 gpd are bled off to a high-rate trickling filter treatment plant. At the trickling filter plant the average phenol concentration in this influent is 8.0 mg/l, with an average reduction of 99.7 per cent. In the other plant, the phenol is totally destroyed by oxidization with chlorine. A typical day's data taken from the operating log is shown in Table I.

This is a report of bench-scale and pilot-scale studies performed for the purpose of finding an economical method for the treatment of phenol waste.

Other Work

There are numerous literature references to the tolerance or development of slimes in a phenol environment. Activated sludge appears to be able, for a time, to handle modest concentrations of phenol and in this respect performs a little better than trickling filters (1). Various investigators re-

TABLE I.—Operating Data from One Day of Treating Phenol Waste for Waste Flow of 10,000 gal; Caustic Soda Used, 150 lb; Alum Used, 40 lb; and Chlorine Used, 160 lb

Item	Influent	Primary Eff.	Final Eff.
Phenol, mg/l	217	205	0.0
pH	9.5	6.8	8.0
Oil, mg/l	280	39	—

TABLE II.—Phenol and BOD Removal

Method of Treatment	Removals		Phenol in Effl. (mg/l)
	Phenol (per cent)	BOD (per cent)	
Emulsion breaking, skimming, and neutralization	40–60	—	400–1,000
Specific biological flora	90–93	94–95	12
Chemical coagulation	60–65	86–87	120
Trickling filter	60–80	70–80	14–8
Activated sludge	99	80–90	Trace

port that appreciable time appears desirable for acclimatization. Many bacteria in pure culture were found to be able to live in phenol concentrations up to 200 mg/l, but few could survive above 900 mg/l. Note was taken of certain strains of aerobes that could be grown in concentrations up to 3,700 mg/l while anaerobes managed up to 4,800 mg/l (2). Some bacteria, such as *Nacardia rubis,* utilized phenol as a sole source of carbon but required additional nutrition, and generally a 0.1-per cent solution of a commercial fertilizer was used (3). Increases in temperature appeared to favor bacterial growth and concentration increases were not detrimental if the increments were kept in the approximate range of 100 to 200 mg/l (2) (3) (4) (5). Summarizing, most of the studies indicating use of phenol as a food in high concentrations were in pure culture, using pure phenol, and with supplementary nutrients.

Earlier studies (6) dealt with the treatment of mixtures of sewage and low concentrations of phenol, particularly with reference to trickling filters. The results are summarized in Table II.

Bench-Scale Studies

The general method of approach in the bench-scale studies was to start with a sewage slime and gradually add phenol and nutrient in increasing dosages insofar as the slime would continue to prosper, then switch to industrial wastes containing phenol. The basic problem was to determine if a specific biological slime could utilize a trade waste as a source of food.

Using 13 experimental setups, phenol loadings on established attached and free floating slime were gradually increased from 2 to 8,500 mg/l over a period of 14 months, with a breakdown concentration of 30,000 mg/l. Loadings were instituted on both a batch and continuous basis.

Apparatus and Techniques

Two basic types of apparatus were developed for the growth of slimes. One was a rotating drum, the other an aeration tank (Figures 1 and 2). The rotating drum had been used in earlier studies (7). The drums were constructed of frosted glass, 4 in. in diam, and 2 in. wide, and revolved at approximately 4 rpm, partially submerged in sewage. The slime grown on these drums proved to behave as

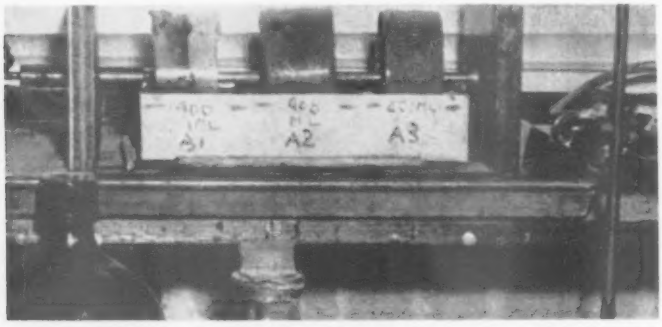

FIGURE 1.—Laboratory rotating drum apparatus.

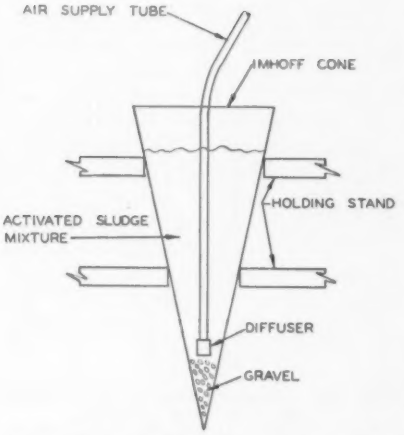

FIGURE 2.—Laboratory activated sludge apparatus.

does the slime on a trickling filter and the reaction coefficient as determined by P^{32} uptake agreed with accepted values for a trickling filter. The aeration tanks consisted of Imhoff cones containing two-thirds sewage and one-third activated sludge to start. Air was supplied at the rate of about 2,000 ml/min.

The method of operation was on a batch basis wherein the tank or cone was filled with sewage, phenolic waste, and nutrient, and then exposed to the slime on the drum or suspended in the waste until the phenol was removed. The waste was then removed from the drum apparatus or decanted from the cone and the cycle was started again.

Most of the data were collected from studies using four drums and nine aeration cones, but modifications were necessary to approximate continuous conditions and to determine reaction coefficients. To simulate continuous flow, nutrients, phenolic waste, and make-up water were fed by an intravenous-type apparatus at a rate such that the concentrations of these elements were the same at all times. This was done on both the drum and aeration equipment. To determine reaction coefficients, a Geiger-Mueller tube was placed over the top of the slime attached to the rotating drum, and after a small amount of P^{32} had been injected into the waste, the reaction coefficient was determined from the accumulative percentage P^{32} uptake.

The phenolic waste used was from Tinker Air Force Base, Oklahoma City, Okla.

Experimental Procedure and Results

The initial studies were operated on a batch basis using the drum and aeration cones with domestic sewage and increasing dosages of phenol. The end point of the batch treatment was determined by the absence of phenol, and the incremental increases in phenol concentration seldom exceeded 50 mg/l. Nutrients were added, scaled on the basis of 0.1-per cent concentration of 12-24-12 fertilizer at 4,000 mg/l of phenol. Three drums and six aeration cones were employed for these studies.

A great variety of experimentation was made with the two basic arrangements. Phenol concentrations as high as 7,500 mg/l were obtained on the drum and 4,200 mg/l in the aerated slime with complete removal (Figures 3 and 4). Wastes containing as high

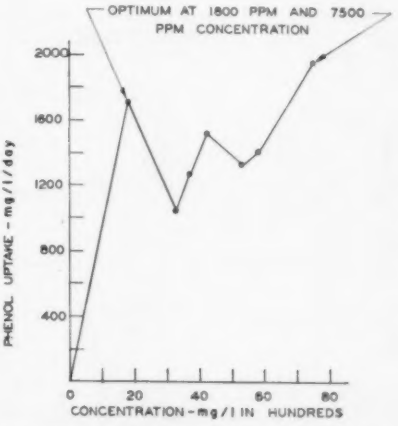

FIGURE 3.—Optimum phenol untake in drum apparatus.

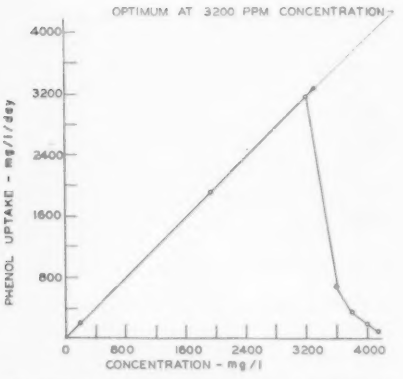

FIGURE 4.—Optimum phenol uptake in aeration apparatus.

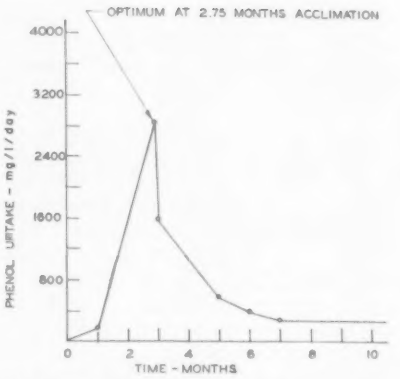

FIGURE 6.—Acclimation period to phenol waste in aeration apparatus.

as 3,200 mg/l phenol were reduced in a shorter period of time, from 6 to 8 hr, by the aeration tanks. The acclimation of the slimes was generally achieved in two to three months (Figures 5 and 6). The slime was closely observed for physical changes, and reaction coefficients were determined by using radio phosphorus. The number of species of bacteria and protozoa decreased, as did k, as the concentrations increased. The k factor was found to be around 0.08, or about one-half of that of sewage slime, at a phenol concentration of 2,000 mg/l. In general, it was found that at least two hours' contact was necessary before any appreciable phenol reduction occurred. Microscopic examination indicated a heterogeneous flora and fauna. These analyses also indicated a shift to homogeneous growth as did color development and development of poor settling characteristics with increasing concentration. Generally, it can be concluded that drum-grown slimes, though they remove phenol more slowly, will tolerate higher concentrations. It is shown in Figure 3 that the optimum concentrations for most efficient removal are at 1,800 and 7,500 mg/l. It was necessary to add fertilizer for the heavier loadings.

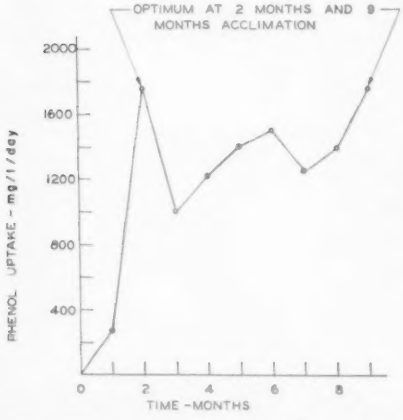

FIGURE 5.—Acclimation period to phenol waste on laboratory drum apparatus.

Substitution of trade waste for pure phenol did not alter the slime growth or phenol waste removal rate.

To stimulate a continuous loading condition as would be encountered in a biological treatment plant, phenol and food concentration were maintained constant, and continuous loadings of 1,000 mg/l on the drum and 2,000 mg/l in the aeration cones were handled satisfactorily.

Pilot-Plant Studies

The next problem was the translation of the laboratory studies to the pilot-plant stage. The pilot plants are

located at Tinker Air Force Base sewage treatment plant. They consist of (*a*) a trickling filter, 4 ft in diam and 6 ft deep, equipped for recirculation and designed to operate as a conventional high-rate filter; (*b*) a rotating drum pilot plant with a drum 15½ in. in diam and 27 in. long, and arranged so that both inside and outside surfaces are used; and (*c*) an activated sludge tank and clarifier of 163- and 40-gal capacity, respectively (Figures 7 and 8).

Actually, the pilot plant has two distinct parts: (*a*) equalization and clarification tanks; and (*b*) biological treatment. Emulsion breaking, skimming, and neutralization were still necessary to prepare the waste for biological treatment.

Chemical Treatment

The waste was transported to the plant and stored. The flow of waste was from the outside tanks to a chemical and air tank where the oil was removed, then to one of the three pilot plants. The chemical treatment consisted of adding, on the average, 410 mg/l $Fe_2(SO_4)_3$, 1,026 mg/l $Ca(OH)_2$, and about 20 mg/l H_2SO_4. The effluent was clear. The pH was lowered to 3.0 or 3.5 with H_2SO_4, then brought up to above 8.0. Coagulant and air were injected, and the waste was skimmed and settled. This treatment unit was a

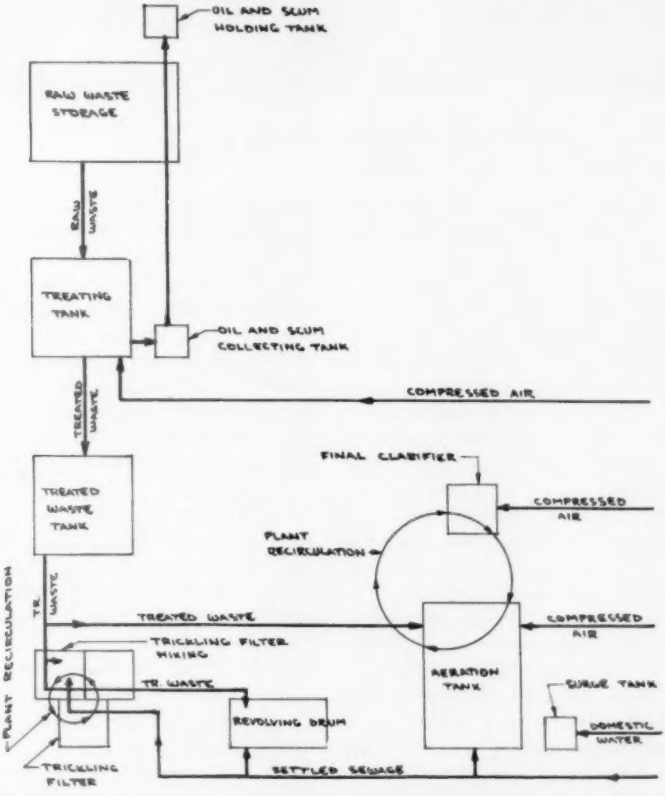

FIGURE 7.—Flow diagram of industrial wastes pilot plant at Tinker Air Force Base.

FIGURE 8.—Pilot plant equipment showing activated sludge units in left foreground and the trickling filter in upper right.

286-gal tank. As much as 35-per cent reduction in phenol concentration was experienced.

Trickling Filter Treatment

In the trickling filter treatment process, primary settled sewage was piped to a small holding tank and passed over a weir to a pump suction well. Clarified phenol waste was introduced to the suction well from a clarified waste holding tank. The phenol concentration was determined for each batch of clarified waste; the flow was controlled by a needle valve and was proportioned to the sewage flow to obtain the desired phenol concentration. The mixed sewage and phenol waste was then pumped to a filter distributor and the rate of flow to the filter was controlled by a valve in the pump discharge and a constant-head glass piezometer tube. Recirculation was maintained by introducing a portion of the filter effluent to the pump suction well. The recirculation was discontinued over weekends due to the very weak sanitary sewage occurring during those periods. The normal flow rate was 4.3 gpm, and the design loading rate was approximately 2 lb BOD/cu yd. With phenol concentrations below 50 mg/l, 80- to 90-per cent removal was obtained with only about 50-per cent recirculation. As phenol concentrations were increased to 100 mg/l, more recirculation was required and the reduction was lowered to about 40 per cent. Despite greatly increased recirculation (90 per cent), only about 10 per cent of an applied 300 mg/l was being removed. This method apparently does not supply sufficient contact time at strong concentrations.

Two filters were used in the study. There was not a bench-scale relationship available for the trickling filter. It was tested because of its common usage and possible combination treatment of wastes highly diluted

with domestic sewage. In these studies the proportion of waste rarely exceeded 10 per cent, the sewage and recirculation accounting for the balance.

For 5½ mo, the rotating drum plant received phenol waste ranging in concentration from 50 to 400 mg/l and chromium from 1.0 to 10 mg/l. The initial retention time in this plant was 2 to 3 hr, with a 60- to 70-per cent removal. After two months operation the removals began to decline and the retention time was increased to 5 hr, but the removals declined to 30 per cent. When the retention time was lowered to 3 hr the removal dropped to 12 per cent. The deterioration in performance may have been caused by the accumulation of chromium in the slime. This same result was noted in the laboratory plant when chromium was present in the waste. The loadings were comparable to the laboratory pilot plants, averaging about 0.008 lb/day/sq ft of slime area.

Activated Sludge Treatment

The activated sludge plant operated with a phenol waste similar to that used in the trickling filter studies. However, the sewage, phenol waste, and return sludge were proportioned and mixed in a funnel at a higher elevation than the aeration tank effluent weir to obtain gravity flow. The mixed liquor flowed from the aeration tank by gravity to the suction of an air lift and was returned to the clarifier. The return sludge was pumped by gravity to the aeration tank influent. In the activated sludge plant, phenol rarely exceeded 10 per cent; the return sludge, 40 per cent, with fresh settled sewage constituting the balance.

In this operation it was not necessary to waste sludge. The DO was kept at about 1 to 2 mg/l, the solids in the mixed liquor, 2,000 to 3,000 mg/l, and the sludge by volume at 40 to 50 per cent. The sludge volume index was about 115 to 150. The removals never were below 89 per cent, and frequently were 100 per cent, while the average was 95 to 96 per cent.

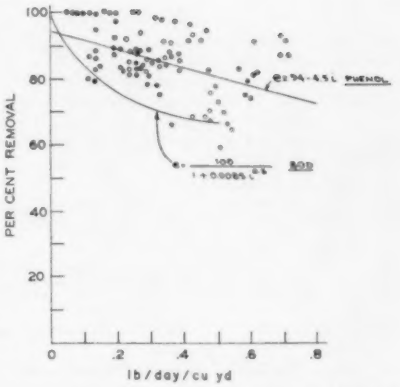

FIGURE 9.—Performance of trickling filter handling phenol waste.

Discussion

The operational data for all three plants are shown in Figures 9, 10, and 11, on the basis of the NRC established technique. The formula is:

$$e = \frac{100}{(1 + mi^n)} \quad \ldots\ldots (1)$$

where m and n are empirically determined constants, i the loading rate, and

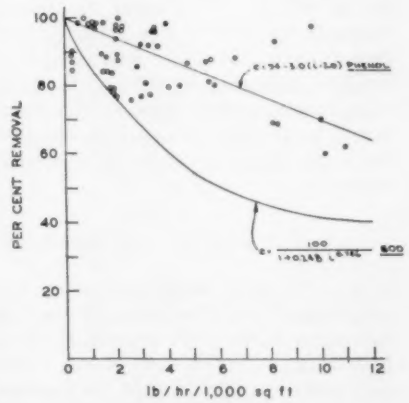

FIGURE 10.—Performance of rotating drum handling phenol waste.

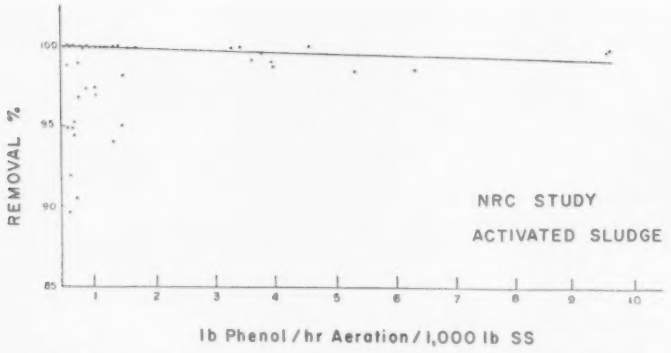

FIGURE 11.—Performance of activated sludge handling phenol waste.

e the removal. Instead of lb of BOD as is done in the NRC work, lb of phenol are used. Total BOD values were not available for comparison, but established values were placed on the curves for comparison.

Because of the obvious inability of the trickling filter to handle strong wastes, even with greatly increased recirculation, recirculation factors were computed:

$$F' = \frac{I + R}{I} \quad \ldots\ldots\ldots (2)$$

where I is the waste plus sewage, and R is the recirculation.

For a recirculation of 90 per cent, which was necessary, F would be 10, or in effect a theoretical 10 passes through the filter. The actual retention time in the filter was computed by dye studies to be 60 sec; theoretically, the retention time at this level would be 10 min. The actual effective recirculation factor is much lower and equal to:

$$F = \frac{I + R/I}{1 + (1 - f)R/I^2} \quad \ldots\ldots (3)$$

with f being a weighing factor approaching 0.9 for trickling filters. A maximum value of F is reached when $R/I = (2f - 1)/(1 - f) = 8.0$. The implication herein is that no further recirculation above 8 is of any biological value. Considering the effective recirculation and actual contact time measurements, the previously reached conclusion regarding trickling filters and concentrated wastes are strengthened.

The computed F' and F values ranged for all the data from 1.3 to 25.0 and 1.2 to 3.0, respectively. The highest loading was 1.04 lb phenol/day/cu yd. These data represent only a partial load on the filter in that BOD is contributed by the sewage. The sewage and phenol in terms of BOD can be interrelated. In this respect Heukelekian and Rand (5) found about 2 g BOD per g of phenol. Correlating BOD and phenol relationships were also obtained for the pilot plants; these factors ranged from 4.0 to 2.0 g BOD/g of phenol. The higher values accounted for the increased volume of sewage.

At loadings of from 22 to 450 mg/l or 34 to 210 lb/1,000 cu ft of aeration capacity, 98 to 100 per cent of the phenol was removed. Calculated on a BOD basis, this would be from 68 to 420 lb BOD/1,000 cu ft aeration capacity for the mixture.

Conclusions

Laboratory and pilot studies indicate that phenol can be treated success-

fully by biological slimes in concentrations as high as 7,500 mg/l. A rotating drum type plant is the best for very high concentrations of phenol. Although the rotary drums are an effective treatment device, they are not preferable to activated sludge or the aeration setups.

The pilot plant studies demonstrate that trickling filters will operate on a sewage and phenol mixture, if the phenol concentration is kept below 100 mg/l, but that it is not adequate for concentrated wastes.

Activated sludge treatment, using NRC standards, retention time, and sludge volume criteria, was very successful for phenol concentrations up to 500 mg/l and BOD of 1,000 mg/l. In this process sludge was not accumulated.

Acknowledgments

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OPPORTUNITY IN WISCONSIN

The City of Fond du Lac is trying to employ an individual for the position of Sewage Treatment Plant Superintendent. The treatment plant includes high-rate trickling filters, separate sludge digestion, vacuum filtration, incineration of sludge, and garbage incineration. The position requires general supervision of city garbage and refuse collections and street sanitation.

A civil or sanitary engineering graduate with previous operation experience is desired. The salary range is $600–$720 per month with liberal fringe benefits. An applicant may obtain further information and application forms from the Personnel Office, City Hall, Fond du Lac, Wisconsin.

Stream Pollution

FORECASTING HEAT LOSS IN PONDS AND STREAMS *

By C. J. Velz and J. J. Gannon

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Disposal and dissipation of heat in streams, lakes, and ponds associated with industrial and utility operations is of increasing importance in stream sanitation. Large quantities of condenser and cooling water used in the modern fuel power generating stations, nuclear reactors, and industries place a heavy demand on water resources. In the ordinary sense of the term pollution, these cooling and condenser waters are returned to the water course unimpaired in quantity and quality. However, during periods of drought the enormous waste heat loads may raise the temperature of the water course substantially above normal levels. This affects the natural waste assimilation capacity of the stream and the normal aquatic life, and may interfere with subsequent downstream uses.

In order to evaluate in quantitative terms the effects of waste heat load in any specific situation, it is necessary to have a rational method of forecasting heat dissipation and the corresponding water temperature profile expected along the course of the receiving stream. This paper deals with such a method, based on conventional stream gaging data and weather records. The meteorologic and hydrologic variables involved are defined in statistical terms, making it possible to assign a probability of occurrence to any forecast of water temperature profile developed. Two practical applications are illustrated: (*a*) use of a wastewater storage lagoon as a cooling pond for reuse of the water and (*b*) use of a river for dissipation of waste heat.

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

Influence of Heat on Stream Sanitation

Waste Assimilation Capacity

Temperature of river water is a major factor in organic waste assimilation capacity of a stream as it plays a triple role, i.e., affecting the rate of stabilization, the dissolved oxygen saturation capacity of the water, and the rate of reaeration. As water temperature rises, the aerobic rate of satisfaction of organic BOD increases, with resultant greater demand on the dissolved oxygen of the river water. The normal amortization of BOD debt at 20°C is at a rate of 20.6 per cent per day of what remains each succeeding day (1); increasing to about 40 per cent per day at a temperature of 40°C. The temperature coefficient governing this increase is generally accepted as 1.047, effective in the range 9 to 30°C. Recent studies indicate that the coefficient is also reasonably applicable in the range 30 to 40° C, but further research is needed at higher temperatures.

Some investigations (2) have shown that in addition to increase in rate of BOD reaction with increase in temperature, there is also an increase in the ultimate demand of the order of 2 per cent per degree centigrade. In recent laboratory studies and river investigations, a significant increase in ultimate demand with the temperatures in the range 20 to 35°C was not found. This is also confirmed by the work of others (3).

In dealing with organic sludge deposits (4), the semi-anaerobic rate of satisfaction of BOD of the sludge is also increased with rise in temperature, thereby increasing the demand on the dissolved oxygen of the overflowing water. A secondary effect is the influence of temperature on the equilibrium accumulation of BOD attained in the deposited sludge. As temperature rises the equilibrium accumulation declines and, conversely, as temperature declines BOD accumulates to higher equilibrium levels. Thus, a sudden rise in temperature following a protracted period of accumulation establishes a combination of high rate of satisfaction acting on an excess BOD accumulation which may induce an abnormally sharp decline in dissolved oxygen.

Unfortunately, as the rate of demand on oxygen resources increases with temperature rise, the dissolved oxygen capacity of water declines from 9.17 mg/l at 20°C, to 7.63 mg/l at 30°C, and to 6.6 mg/l at 40°C. Recent work indicates that the long accepted saturation values are somewhat high (5) (6) (7).

In the complex phenomenon of reaeration, temperature plays a dual role, by influencing the rate of solution which is governed by the oxygen saturation deficit, and by affecting the rate of diffusion of oxygen within the body of water. The temperature coefficient is not firmly established, ranging from 1.1 to 1.047. Thus, as temperature increases, the saturation capacity declines; however, opposing this, as temperature increases the rate of diffusion increases. Where ice cover is a factor, heat loads which maintain open water for reaeration, of course, are a valuable asset to waste assimilation in severe climates.

While in most situations the net effect of increase in river temperature is a reduction in organic waste assimilation capacity, the opposing interrelations require careful evaluation in each specific situation, particularly where changing temperature profiles induced by waste heat loads are involved.

From the standpoint of bacterial self-purification, the effects of temperature on coliform concentration below sewage effluents and on downstream survivals are less firmly established. The phenomenon of aftergrowth, in part, is influenced by temperature, generally increased with increase in temperature (8). The early or first stage of the death rate curve, following the initial aftergrowth, is logarithmic in character, as with BOD. However, with the present state of knowledge, temperature effects on the death rate can be defined only in broad terms; under warm weather temperatures (15 to 30°C) 70 to 85 per cent of the coliform organisms die per day of those surviving on each succeeding day, and at cold weather temperatures (0 to 10°C) 55 to 65 per cent die per day. Where intensive river sampling is available, these rates are in reasonable agreement with observed declines in coliform concentrations.

Aquatic Life

Various species of fish and the aquatic plants and animals on which they feed are affected by temperature of their water environment and each has its tolerance range. Moderate temperature rise may not be injurious, and for some organisms may actually result in increase in population. But sudden increase in temperature or rise beyond the tolerance range may kill

aquatic life or reduce populations. Elimination of one species in the food chain may upset the entire ecological balance.

Increase in temperature affects the oxygen requirements and minimum concentration for survival. Generally, rise in temperature stimulates the metabolic rate, thereby increasing oxygen utilization. The tolerance limits of substances toxic to fish and aquatic organisms are usually lowered by increase in water temperature, rendering the fish more susceptible. High temperatures associated with organic waste sources may induce serious secondary nuisances associated with heavy slime growth or aquatic weeds.

Some aspects of the effects of heat loads on aquatic life can be reasonably forecast, but frequently under apparently similar circumstances quite different end results develop. Detailed "before" and "after" field studies and further laboratory research are needed for a better understanding of biological thermodynamics.

Water Reuse

With the trend toward large industrial-urban concentrations and associated power developments, competition for limited water resources is intensified and the river water is used over and over again. In such settings, the impact of one user on another is compounded. Before the increment of heat load from one source is dissipated, another is added, until water temperature rises to such an extent as to interfere with subsequent use or to render the stream practically useless for any other purpose. During the drought season a number of basins in the United States approach these conditions.

Sources and Magnitude of Heat Load

Quantitative data concerning waste heat loads discharged to streams and the proportion assignable to utility, industry, and municipal sources generally are not available. Detailed studies on the Thames Estuary (9) show that waste heat load from steam electric power generation accounts for 74.5 per cent of the total, with all other sources aggregating 25.5 per cent. With the trend toward large, fuel-generated stations, condenser circulating water systems of steam power plants are likely to remain the largest single source of waste heat.

It is estimated (10) that approximately one-half of all the heat generated is dissipated in the condenser circulating water system and is discharged as waste heat to the receiving stream. The heat loads from specific installations vary widely depending on design and operation. In general, the smaller generating units produce higher unit waste heat loads and require larger unit quantities of circulating water. Sargent and Lundy (11) have generalized these relationships in graphic form as shown in Figure 1. Curve *A* shows the relation between the unit capacity in kw and the circulating water requirements in gpm/kw; Curve *B* shows the relation between the unit capacity in kw and the

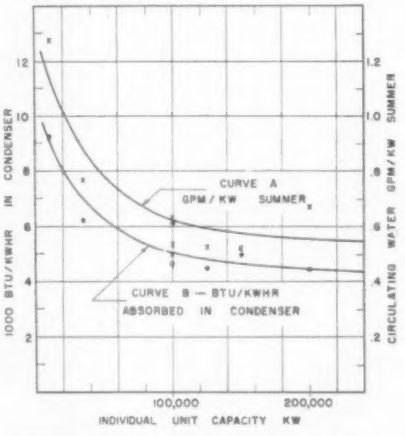

FIGURE 1.—Steam power plant condenser characteristics in river plants with two-pass condensers (11).

heat discharged in Btu/kwh. It should be pointed out that capacity refers to individual unit capacity and not the total plant capacity and that the circulating water requirements refer to summer conditions. An inspection of the graph shows an inverse curvilinear relationship between unit size and the heat load and the circulating water requirements, leveling off to a heat load of about 4,400 Btu/kwh, and circulating water needs to about 0.55 gpm/kw. For a generating unit of 100,000 kw capacity, the heat load from Curve *B* is about 5,000 Btu/kwh, and the circulating water requirement from Curve *A* is about 0.6 gpm/kw, for summer conditions. During the winter period the water requirements have been reported to be about 60 per cent of the summer requirements. The data reported in Figure 1 appear to be in reasonable agreement with that reported by others.

Meteorologic and Hydrologic Factors Influencing Water Temperature

One of the early attempts to establish a numeric basis for predicting the cooling effects of ponds was that of Lima (12). His relationships were established empirically from data collected at existing installations, and centered around a coefficient K which was related to air temperature under standard conditions of a 6-mph wind. Another method for predicting the cooling effects on ponds was proposed by Thorne (13), employing a modified energy budget relationship based on observations made in Colorado.

LeBosquet's pioneering work (14) presents a mathematical basis for predicting heat loss in a flowing stream. The factors taken into consideration are the river flow, the differential between water and air temperature, river channel characteristics, and a heat loss coefficient K. Unfortunately, it is essential to derive K from an observed river temperature profile. The coefficient varies widely from river to river; according to LeBosquet, this variation is from 6 to 18. In studies of the Ocmulgee River in Georgia, based on an observed river temperature profile, K was determined as 10.4 Btu/hr/sq ft/°F differential. LeBosquet's equation takes a semi-logarithmic form, convenient in application, and where K is known, predictions can be made for other heat loads and runoff conditions.

More recently, British workers (9) (15) in studies of the effects of waste heat in the Thames Estuary and the River Lea, propose an empirical approach involving a proportionality constant f similar to LeBosquet's constant K, which also must be derived from an observed river temperature profile. Based on intensive observed data on River Lea, the constant f was determined as 5.4 Btu/hr/sq ft/°F differential.

The most complete attempt to investigate the various terms of the energy budget of water was made at Lake Hefner, Okla., by a combined Federal task force, including the U. S. Navy, Bureau of Reclamation, Geological Survey, and Weather Bureau (16). While the primary purpose was to investigate the phenomenon of evaporation from water surfaces, much basic information was assembled on heat loss. For conditions similar to those of Lake Hefner, it was found that substantial savings in water can be realized by using a reservoir or natural lake rather than cooling towers (17).

Based on energy budget relationships, Langhaar (18) proposed a method for predicting cooling effects of ponds which permits direct computation without dependence on observed water temperatures. Modification of Langhaar's basic approach, as developed herein, is the method believed to be most applicable to evaluation of waste heat problems.

Derivation of Basic Equations

In the development of basic equations (following the terminology of Langhaar) four principal surface mechanisms are involved in heat transfer and temperature change of a body of water exposed to the atmosphere. Heat is lost through evaporation, convection, and radiation and is gained through solar radiation. Heat may also be transferred or lost by infiltration to and into the enclosing earth bottom, but in a flowing stream and in well-sealed basins this may be neglected. The heat balance is represented as

$$H = H_e + H_c + H_r - H_s \ldots (1)$$

where

H = net heat loss,
H_e = heat loss by evaporation,
H_c = heat loss by convection,
H_r = heat loss by radiation, and
H_s = heat gain by solar radiation.

The three heat losses: evaporation (H_e), convection (H_c), and radiation (H_r) can be computed from meteorologic measurements of air temperature, wind velocity, and vapor pressure.

Evaporation Loss

Heat loss by evaporation (H_e) is readily calculated by Meyer's (19) formula, which has wide acceptance in engineering practice and is particularly applicable because it is based on routine U. S. Weather Bureau observations. Meyer's formula for evaporation I, expressed in inches per month, from natural bodies of water is

$$I = C(1 + 0.1W)(V_w - V_a) \ldots (2)$$

where W is mean wind velocity in miles per hour measured about 25 ft above the water surface or above the surrounding land area; V_w is vapor pressure in inches Hg corresponding to surface water temperature taken about 1 ft below the surface; V_a is the mean absolute vapor pressure prevailing in the overlying atmosphere 25 ft above the water surface or above the surrounding land area; and C is a constant, ranging from 10 to 15, depending on the depth and exposure of the body of water and frequency of meteorologic measurements. For large deep lakes and reservoirs, C is taken near the lower value, and for shallow ponds and surface accumulations, C is taken as 15. For flowing streams of moderate depth and velocity, C may be taken as 14. Using latent heat of vaporization (H_v) for the given water temperature, I, in inches per month, is converted to H_e, in units of Btu per hour per square foot of water surface, by

$$H_e = 0.00722 H_v C \times (1 - 0.1W)(V_w - V_a) \ldots (3)$$

Convection Loss

Heat loss by convection depends on the temperature differential between the water and air and on the wind velocity at the surface of the water. In still air, convection loss from a flat surface ranges from 0.5 Btu/hr/sq ft/°F for a temperature differential of a few degrees to 1.0 for temperature differentials to 50 to 100°F. For usual cooling pond situations, Langhaar suggests a coefficient of 0.8. Air movement at the surface is reported to increase the coefficient by 0.16 to 0.32 for each mph of wind velocity. For a quiescent body of water such as a cooling pond, Langhaar suggests an average value of 0.24. For flowing streams of moderate velocity the higher value approaching 0.32 would be reasonable. The greatest factor of uncertainty is the conversion of nearby U. S. Weather Bureau wind record to velocity at the surface of the water. This depends on relative exposure and usually is taken as half that recorded at official Weather Bureau stations.

Convection loss for a flowing river then is given by

$$H_c = \left(0.8 + 0.32\,\frac{W}{2}\right)(T_w - T_a) \ldots (4)$$

where T_w is the water temperature at the surface and W and T_a are Weather Bureau measurements of mean wind velocity in miles per hour and mean air temperature, respectively.

Radiation Loss

Heat loss by radiation from the water surface (H_r) depends on the temperature differential between the water and the air as well as the radiation from surrounding objects and terrain. Assuming surrounding objects and terrain at the same temperature as the air and an emissivity of 1.0 from the water surface, the net heat loss by radiation for ordinary temperatures is given by

$$H_r = 1.0(T_w - T_a) \quad (5)$$

Solar Radiation Gain

Heat gain by solar radiation (H_s) cannot be computed from meteorologic factors and must either be estimated, or preferably, measured. Determination from maximum possible sunshine leads to values too high, as local sky cover radically reduces the radiation reaching the water surface. Local effects such as shade by trees along river banks or community and industrial air pollution also can reduce substantially solar radiation reaching the water surface. The absorption coefficient of water surface is high, exceeding 95 per cent; hence the controlling factors are local effects. The measured solar radiation, direct and diffuse, on a horizontal surface as determined at nearby Weather Bureau stations is considered directly applicable if exposures are reasonably similar.

Equilibrium Water Temperature

A body of water exposed to a given set of meteorologic conditions, with time, will come to equilibrium such that the heat losses by evaporation, convection, and radiation will balance the heat gain from solar radiation. Under equilibrium condition net heat loss is zero, and from Equation 1,

$$H_e + H_c + H_r = H_s \quad (6)$$

Under these conditions the temperature of the water will also approach an equilibrium limit. This limiting temperature is referred to as the equilibrium temperature E and can be computed from the measurements of the prevailing meteorologic conditions as follows:

Substituting the three expressions of heat loss in Equation 1 gives

$$H = .00722H_vC(1+.1W)(V_w - V_a) + (.8+.16W)(T_w - T_a) + 1.0(T_w - T_a) - H_s \quad (7)$$

At equilibrium the temperature of the water $T_w = E$, and $V_w = V_E$. Substituting and combining gives an equation for E and V_E in terms of the measurable meteorologic factors.

$$(1.8+.16W)E + .00722H_vC(1+.1W)V_E = (1.8+.16W)T_a + .00722H_vC(1+.1W)V_a + H_s \quad (8)$$

A direct solution for E cannot be made by Equation 8 as V_E is also involved, but by assuming values for E by successive approximations a balance in the equation is quickly approached which thus determines the proper value of E.

Heat Dissipation and Water Temperature Profile

Theoretically, an infinite time of exposure is required for water to attain equilibrium with given sustained meteorologic conditions. Also, an infinite surface area would be required for warm water introduced into a river or basin to cool to the equilibrium temperature. Fortunately, the temperature decline is nearly logarithmic. Thus, while complete cooling to equilibrium temperature is not fully attainable, close approach to that level is obtained within practical limitations of time, basin area, or river reach.

The relation between water temperature profile and required surface cooling area involves the following concepts of net heat loss H based on observed meteorologic conditions. An increment of water temperature decline dT_w per increment of time dt is equal to the heat loss from the corresponding increment of surface area divided by the weight of the underlying water of b depth.

$$\frac{dT_w}{dt} = -\frac{H}{62.4b} \quad \ldots\ldots\ldots (9)$$

Integrating gives

$$\int_{T_1}^{T_2} dT_w = -\frac{H}{62.4b}\int_{T_1}^{T_2} dt \quad \ldots (10)$$

Expressing t in hours, in terms of area A in square feet per cfs of stream flow,

$t = \dfrac{bA}{3{,}600}$, and solving for A gives

$$A = -224{,}640\int_{T_1}^{T_2}\frac{dT_w}{H} \quad \ldots\ldots\ldots (11)$$

From Equation 6, when $T_w = E$, $H_s = H_e + H_c + H_r$
from which

$$H_s = .00722H_vC(1+.1W)(V_E - V_a) + (1.8+.16W)(E-T_a) \quad \ldots (12)$$

Substituting in Equation 7 gives

$$H = .00722H_vC(1+.1W)(V_w - V_E) + (1.8+.16W)(T_w - E) \quad \ldots (13)$$

Combining Equation 13 with Equation 11 and substituting α for

$$0.00722\ H_vC(1 + 0.1W)$$

and β for $(1.8 + .16W)$ gives

$$A = -224{,}640 \times\int_{T_1}^{T_2}\frac{dT_w}{\alpha(V_w - V_E)+\beta(T_w - E)} \quad (14)$$

While Equation 14 cannot be integrated directly, it is possible to reach solutions, without resorting to empirical procedures, by dividing the temperature differential $T_1 - T_2$ into a number of equal increments ΔT_w and performing the integration by successive summations, taking T_w and its corresponding V_w as the mean for each successive increment in accordance with

$$A = -224{,}640 \times\sum_{T_1}^{T_2}\frac{\Delta T_w}{\alpha(V_w - V_E)+\beta(T_w - E)} \quad \ldots (15)$$

Usually summation of 4 to 8 increments is adequate and since the complete temperature profile is desirable, the successive increments provide the intermediate points between the entering water temperature T_1 and the final temperature T_2.

Thus, where long term weather records are available it is possible by Equation 8 to compute the equilibrium water temperature expected from any probability of occurrence of meteorologic conditions. The equilibrium temperature E represents the limit of cooling possible in a surface pond or stream. In turn, cooling to any level of temperature above equilibrium can be computed by Equation 15. Knowing the cumulative surface area along the course of the stream for the particular runoff from channel cross-sectioning, the river temperature profile for that runoff can be constructed.

Variation in Meteorologic and Hydrologic Factors

In applying the basic equations to a practical situation, of equal or greater importance than the adoption of the coefficients applicable to the local conditions is the decision concerning probability of occurrence of meteorologic and hydrologic factors. Air temperature, wind velocity, vapor pressure of the atmosphere, solar radiation, and stream flow are dynamic variables. While these factors vary radically with time, within the natural setting the variation is orderly and can be defined by statistical methods from long term records of observations reported by the U. S. Weather Bureau and by the U. S.

Geological Survey at or near the location under investigation.

In order to relate a water temperature profile to physical locations below a point of introduced heat load, it is essential to define the channel characteristics of width, depth, surface area, channel volume, and time of passage along the course of the stream. If such data are not available they can be readily obtained in the field by channel cross-section sounding at about 500-ft intervals along the course, employing portable echo sound equipment and recorder. Each runoff regime produces its corresponding channel characteristics. Having field measurements for one runoff condition, based on appropriate stage rating curves, adjustments can be made in channel characteristics for other probabilities of stream flow.

Expected Variation in Meteorologic Factors

Air Temperature

The seasonal pattern of air temperature and the expected deviations from year to year are reflected in the long-term records of the monthly average recorded at U. S. Weather Bureau stations. Statistical analyses of such records, considering each month of the year separately, quite consistently develop normal distributions from which the most probable monthly mean air temperature and the standard deviation are readily obtained graphically (20). Figure 2 represents a typical normal distribution based on a 37-yr record of July monthly averages at the U. S. Weather Bureau station at Midland, Mich. From similar plots for each month a clearly defined seasonal pattern emerges as shown in Figure 3. Curve *A* represents the most probable monthly mean air temperature expected at Midland for each month of the year, and the shaded belt based on the standard deviation represents the range within which monthly mean is expected to fall for 80 per cent of the years. On 10 per cent of the years it can be expected that monthly mean air temperature for specific months of the year will fall above the upper boundary of the shaded belt and that on 10 per cent of the years it will fall below the lower boundary. Any other range desired can be obtained from the probability plots.

Wind Velocity

The velocity of the wind is extremely variable and the geographical coverage reported at Weather Bureau stations is not as extensive as air temperature. The seasonal pattern, again, is best reflected by the monthly averages at stations with continuous long-term records. Statistical analyses of monthly average wind velocities indicate reasonably normal distribution plots for

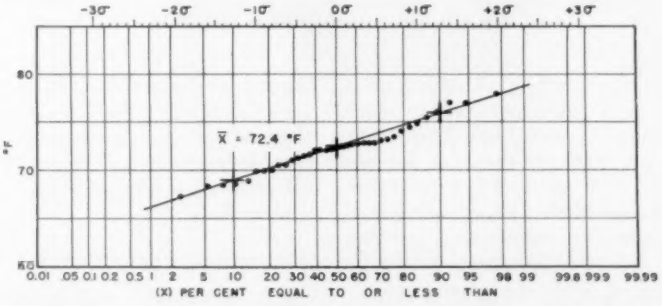

FIGURE 2.—Monthly average air temperature for July at Midland, Mich., based on records from 1920–1956.

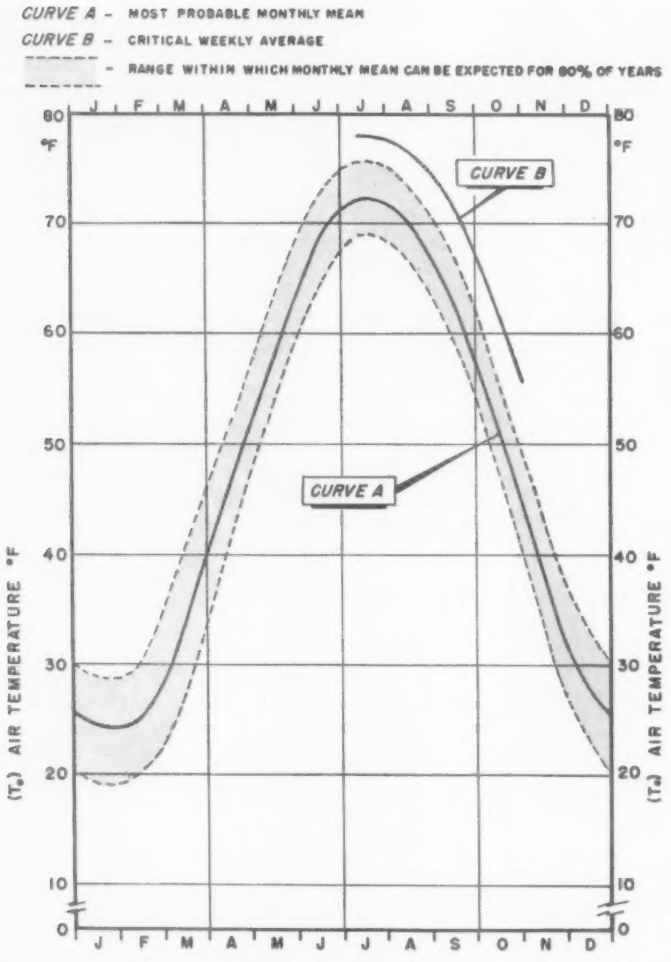

FIGURE 3.—Expected seasonal pattern of air temperature at Midland, Mich., based on records from 1920–1956.

each specific month of the year, from which a most probable monthly mean and standard deviation are determined. Figure 4 represents the plot for the month of July for a 33-yr record at East Lansing, Mich. From such analyses, a reasonably orderly seasonal pattern of variation emerges, as shown in Figure 5, with the most probable monthly mean velocity indicated by Curve *A* and the 80-per cent range indicated by the shaded belt. High wind velocity in this locality is normally expected in the spring, peaking in March; low velocities coincide with the hot summer season, with the minimum in August.

Atmospheric Vapor Pressure

Absolute vapor pressure of the atmosphere in inches Hg, determined from the average of the morning and

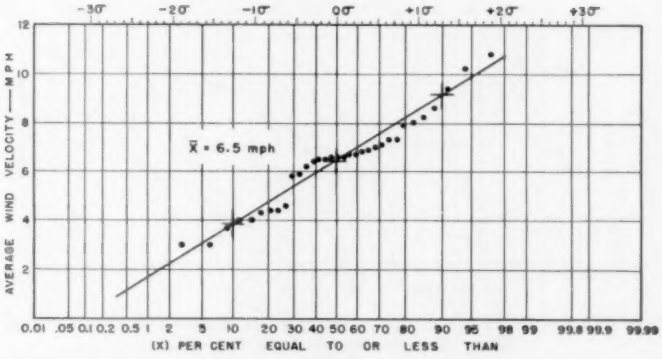
−3σ
−2σ
−1σ
0σ
+1σ
+2σ
+3σ
AVERAGE WIND VELOCITY — M P H
X̄ = 6.5 mph
0.01 .05 0.1 0.2 0.5 1 2 5 10 20 30 40 50 60 70 80 90 95 98 99 99.8 99.9 99.99
(X) PER CENT EQUAL TO OR LESS THAN

FIGURE 4.—Monthly average wind velocity for July at East Lansing, Mich., based on records from 1922–1954.

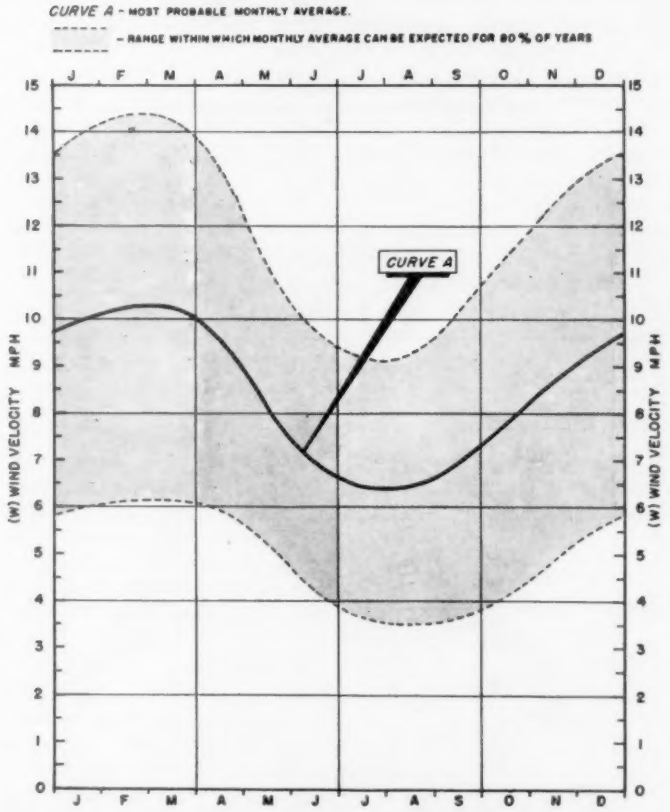
CURVE A - MOST PROBABLE MONTHLY AVERAGE.
- RANGE WITHIN WHICH MONTHLY AVERAGE CAN BE EXPECTED FOR 80 % OF YEARS
J F M A M J J A S O N D
CURVE A
(W) WIND VELOCITY MPH

FIGURE 5.—Expected seasonal pattern of monthly average wind velocity in lower Tittabawassee Basin, based on East Lansing records from 1922–1954.

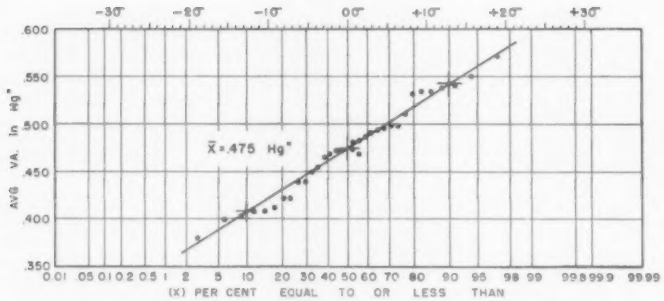

FIGURE 6.—Monthly average vapor pressure for June at East Lansing, Mich, based on records from 1922–1954.

evening monthly mean relative humidity and monthly mean temperature, is employed in developing the seasonal pattern. Again, statistical analyses of the monthly averages, considering each month of the year separately, indicate normal distributions, from which the most probable monthly

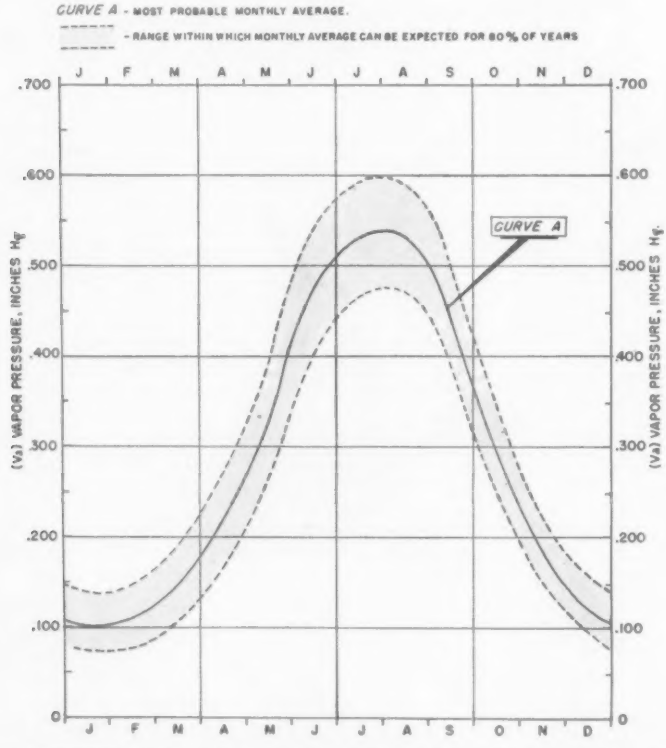
CURVE A - MOST PROBABLE MONTHLY AVERAGE.
- RANGE WITHIN WHICH MONTHLY AVERAGE CAN BE EXPECTED FOR 80% OF YEARS
J F M A M J J A S O N D
(Va) VAPOR PRESSURE, INCHES Hg
CURVE A

FIGURE 7.—Expected seasonal pattern of monthly average vapor pressure for lower Tittabawassee Basin, based on East Lansing records from 1922–1954.

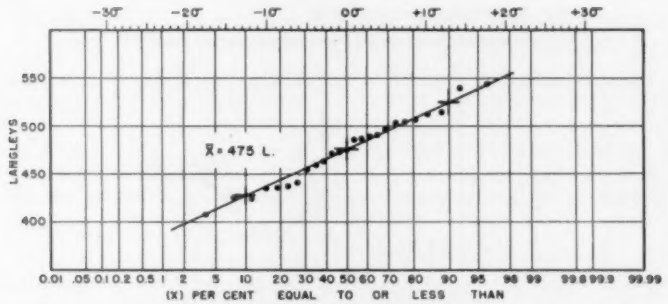

FIGURE 8.—Monthly average solar radiation for August at Madison, Wis., based on records from 1933–1957.

mean and standard deviation for each specific month of the year are determined. Figure 6 is a typical distribution plot for the month of June based on the East Lansing record. A quite orderly seasonal pattern of vapor pressure emerges, as shown in Figure 7, with the most probable monthly mean designated as Curve *A* and the 80-per cent range indicated by the shaded belt. The seasonal peak vapor pressure coincides with the summer warm weather period, July to August, and the low coincides with the winter season.

Solar Radiation

The fourth primary meteorologic factor involved in determining water temperature is solar radiation. Unfortunately, records are available at only a limited number of locations and usually for short periods. As reported, solar radiation reflects direct and diffuse radiation received on a horizontal surface expressed in Langleys, which are converted to Btu per hour per square foot by multiplying by the conversion factor 0.1535.

One of the longest continuous records reported is at the U. S. Weather Bureau station at Madison, Wis., covering the period 1933 to 1957. Dealing with monthly averages for each month of the year, normal distributions develop as with the other meteorologic factors, and Figure 8 is a typical plot for the month of August. In view of the short records available at other locations, of interest is the nature of variation in monthly average solar radiation through the seasons of the year based on the long record at Madison. Table I lists the most probable solar radiation, the standard deviation, and the coefficient of variation based on the plots of the distributions for each month of the year.

As is to be expected, solar radiation is highest during summer, but variation relative to the most probable as reflected by the coefficient of variation is least in mid-summer, increasing to maximum variation in winter.

Where feasible, recourse to records enclosing the area under investigation is advisable. Reasonably normal dis-

TABLE I.—Variation in Monthly Average Solar Radiation Based on Madison, Wisconsin, Record 1933–1957

Month	Most Probable Monthly Mean (Langleys)	Standard Deviation (Langleys)	Coefficient of Variation
Jan.	147	19.0	0.129
Feb.	218	34.0	.156
Mar.	317	40.0	.126
Apr.	402	50.0	.124
May	478	57.5	.120
June	541	46.8	.087
July	559	37.0	.066
Aug.	476	38.5	.081
Sept.	366	45.0	.123
Oct.	267	30.8	.115
Nov.	153	23.8	.155
Dec.	125	27.5	0.220

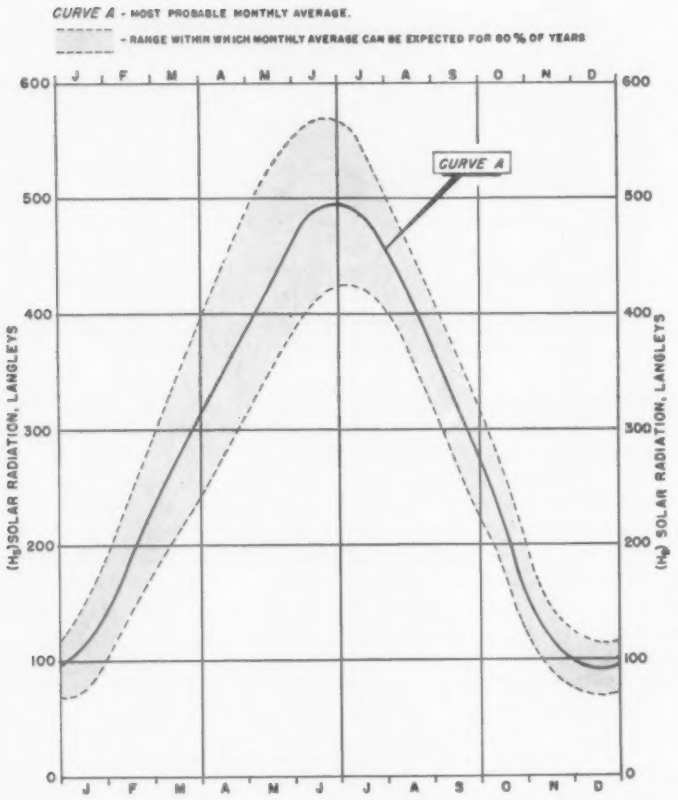

FIGURE 9.—Expected seasonal pattern of monthly average solar radiation in lower Tittabawassee Basin, based on East Lansing records from 1943–1957.

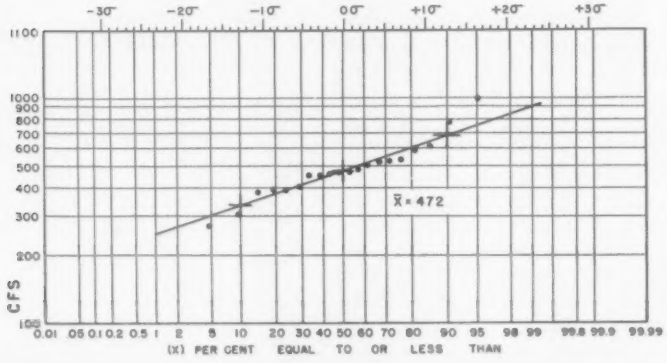

FIGURE 10.—Monthly average runoff for August at Midland, Mich., based on records from 1936–1955.

tributions develop even from short records, from which the local seasonal pattern can be constructed, as illustrated by Figure 9 for the 15-yr East Lansing, Mich., record.

Expected Variation in Hydrologic Factors

Water is a variable and in its utilization in the natural setting, the laws of chance occurrence govern its availability. In addition to very unequal geographical distribution, the quantity of stream flow tomorrow will be different from today, varying through the seasons from year to year, with extremes of flood and drought. While the average flow of a river may appear good, the real restriction limiting the quantity available for use is the uncontrolled natural drought flow. Two statistical evaluations aid in judging availability, the seasonal pattern of runoff and the determination of the drought flow probabilities for the warm weather summer-fall season.

Seasonal Pattern of Runoff

Analyses of long-term monthly average discharge records at U. S. Geological Survey stream gaging stations reflect the seasonal pattern of runoff and the deviations expected. The variation of discharge for any month from year to year is usually great and generally approaches a logarithmically normal

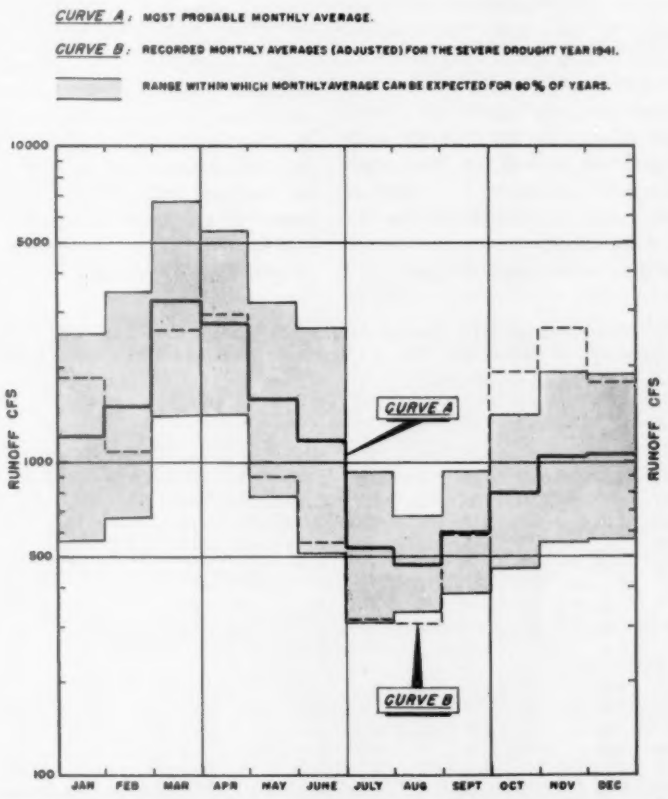

FIGURE 11.—Seasonal pattern of runoff for Tittabawassee River, based on Midland, Mich., gage, adjusted for diversion by Dow Chemical Co.

distribution. Considering each month of the year separately, plotting the monthly averages on log-probability paper readily defines the most probable monthly average discharge and the expected ranges, from which the seasonal pattern can be constructed. Figure 10 illustrates the distribution for the month of August for the Tittabawassee River at the Midland gage. Since the distributions are logarithmically normal, the seasonal pattern is best developed on semi-log paper, as illustrated in Figure 11. Curve *A* represents the most probable monthly average discharge expected, and the shaded belt about the most probable represents the range within which monthly average discharge for specific months of the year can be expected for 80 per cent of the years. The seasonal pattern for any specific year, however, may deviate substantially from the general pattern represented by Curve *A*, as not all months through the year will experience runoff of the same probability of occurrence. Curve *B* shows the monthly hydrograph for the specific year 1941, generally recognized as a year of severe drought.

Drought Flow Severities and Probabilities

Selection of the minimum flow for each warm weather summer-fall season over the years of record constitutes a series of extremes, the distribution of which generally is in close agreement with the theory of extreme values developed by Gumbel (21). The minimum daily average and the minimum calendar monthly average discharges are obtained direct from U. S. Geological Survey water supply papers; however, to develop the entire drought flow pattern it is desirable to select from the summer-fall season for each year, in addition, the minimum consecutive 7-day average, the minimum consecutive 15-day average, and the minimum consecutive 30-day average. As an illustration, Table II represents such series for the Tittabawassee River, Midland gage record (adjusted for diversions), for the summer-fall season May through October, 1938–1955. Figure 12 represents plots of these series on Gumbel log-extremal probability paper in a manner suggested by Velz and Gannon (22). Curve *A* shows the distribution for the minimum consecu-

TABLE II.—Tittabawassee River Drought Flows in cfs Midland Gage Adjusted for Diversions: Summer-Fall Season with Date of Occurrence: Drainage Area 2,400 sq miles

Year	Daily Avg	Date	7-Day Avg	Dates	15-Day Avg	Dates	30-Day Avg	Dates	Monthly Avg	Month
1938	262	July 19	304	7/21–7/27	317	7/13–7/27	364	7/11–8/9	423	July
1939	193	July 20	227	7/16–7/22	270	7/11–7/25	297	7/9–8/7	342	July
1940	251	Sept. 23	303	7/27–8/2	367	7/10–7/24	384	7/9–8/7	471	July
1941	200	Aug. 6	238	8/2–8/8	262	7/12–7/26	290	7/12–8/10	308	Aug.
1942	292	Aug. 29	382	8/12–8/23	418	8/16–8/30	438	8/9–9/7	459	Aug.
1943	262	Sept. 26	448	9/29–10/5	488	9/20–10/4	532	9/12–10/11	564	Sept.
1944	211	Aug. 20	303	8/14–8/20	348	8/7–8/21	375	8/5–9/3	403	Aug.
1945	270	Aug. 26	316	8/20–8/26	372	8/15–8/29	440	7/29–8/27	452	Aug.
1946	236	July 28	286	7/26–8/1	324	7/26–8/9	349	7/26–8/24	390	Aug.
1947	236	Aug. 17	272	8/16–8/22	342	8/9–8/23	375	8/6–9/4	390	Aug.
1948	250	July 26	272	7/25–7/31	300	7/18–8/1	363	8/31–9/29	364	Sept.
1949	185	Aug. 21	261	8/21–8/27	299	8/21–9/4	361	8/6–9/4	368	Aug.
1950	285	Aug. 20	360	9/1–9/7	384	8/24–9/7	410	8/12–9/10	504	Aug.
1951	294	Aug. 12	361	8/8–8/14	393	8/1–8/15	444	7/28–8/26	499	Aug.
1952	349	July 1	447	7/4–7/10	484	7/1–7/15	575	9/28–10/27	579	Oct.
1953	310	Sept. 16	346	8/30–9/5	362	8/28–9/11	395	8/24–9/22	484	Sept.
1954	271	Aug. 26	341	8/21–8/27	362	8/18–9/1	379	8/12–9/10	459	Aug.
1955	297	Aug. 24	344	8/21–8/27	395	8/12–8/26	414	7/29–8/27	499	Aug.

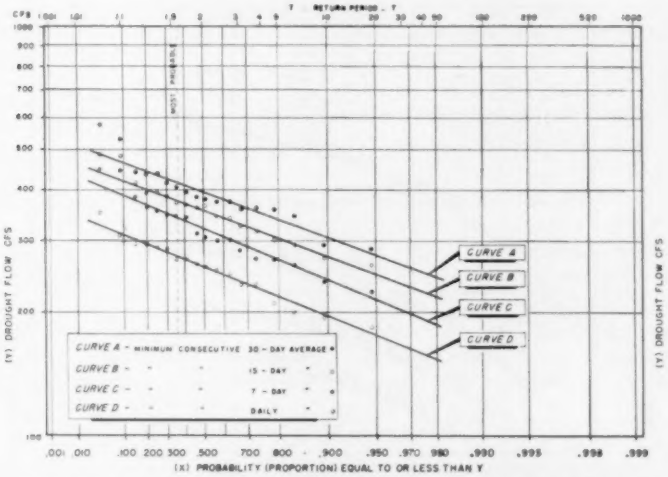
T RETURN PERIOD T
CFS
MOST PROBABLE
CURVE A
CURVE B
CURVE C
CURVE D
CURVE A - MINIMUM CONSECUTIVE 30 - DAY AVERAGE
CURVE B - " " 15 - DAY "
CURVE C - " " 7 - DAY "
CURVE D - " " DAILY "
(Y) DROUGHT FLOW CFS
(X) PROBABILITY (PROPORTION) EQUAL TO OR LESS THAN Y

FIGURE 12.—Drought flow probability for summer to fall period (May–October), based on Midland, Mich., gage records from 1936–1956 and adjusted for diversion by Dow Chemical Co., and addition from Lake Huron.

tive 30-day average drought flows; Curve *B*, *C*, and *D*, the minimum consecutive 15-, 7-, and 1-day average, respectively. From these probability distribution curves it is easy to obtain the drought severity for any probability of occurrence, such as the most probable or the once in 5-, 10-, and 20-yr frequency, as summarized in Table III.

As the period over which the discharge is averaged is shortened, drought severity increases, as shown by the progressive decline in the distribution curves of Figure 12. From the standpoint of pollution and heat dissipation, consecutive low flow for a period equal to the time of passage through the critical reach of the stream is the significant interval. From these distributions, the relation between severity and duration of the period over which drought discharge is averaged is developed, as shown in Figure 13. This graph provides a concept of the entire drought flow structure expected at Midland during the summer-fall period, May through October, as an average over any number of consecutive days from one to 30-days duration; Curve *A* is the most probable drought expected any year, and Curves *B*, *C*, and *D* droughts expected on the average in the long run once in 5, 10, and 20 yr, respectively.

It should be mentioned that in some respects Figure 13 for the Tittabawassee River is atypical as the sharp

TABLE III.—Tittabawassee River Drought Severities Expected at Midland, Mich., Summer-Fall Season (May–October)

Drought Severity	Minimum Daily Average (cfs)	Minimum 7-Day Average (cfs)	Minimum 15-Day Average (cfs)	Minimum 30-Day Average (cfs)
Most probable	274	341	375	414
Once in 5 yr	219	271	306	338
Once in 10 yr	197	242	276	307
Once in 20 yr	177	217	251	278

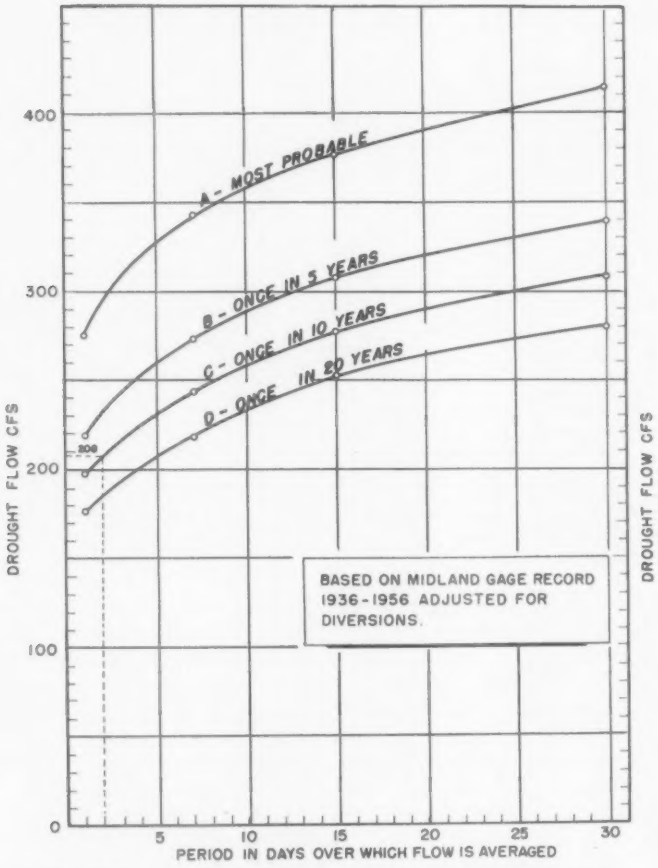

FIGURE 13.—Drought flow characteristics at Midland, Mich.

decline in the curves to the minimum 1-day average discharge reflects upstream hydro power plant operation. In many natural streams the drought flow curves tend to approach straight lines.

Variation in Water Temperature Expected Under Natural Conditions

The statistical nature of meteorologic change induces a dynamic rather than an equilibrium state. Water lags behind meteorologic change, but constantly the shift is toward equilibrium temperature. A stable condition for a month would result in temperatures closely approaching equilibrium. In shallow streams and ponds, deviations from the general seasonal temperature trends can be expected with diurnal cycles. A sustained meteorologic condition for a period of a week may induce 3 to 5°F temporary deviation from the general seasonal trend. Hence, interpretation of observed or computed water temperatures should be made in light of the actual dynamics of the situation with due regard to possibilities of short-term deviations.

Expected Seasonal Pattern of Water Temperature

Since few long-term records of observed water temperature are available from which probability analyses of variation can be determined directly, it is usually necessary to develop the seasonal pattern from computed equilibrium values employing Equation 8. Ideally, equilibrium water temperature should be computed for each month over the period of years for which there are available simultaneous records of air temperature, wind velocity, vapor pressure, and solar radiation. However, the short record of solar radiation available for most areas limits this approach. Also, not all situations warrant the extensive computations entailed in computing water temperatures on a monthly basis.

A review of the observed patterns of occurrence of meteorologic factors suggests a compromise approach in developing the expected seasonal water temperature pattern, retaining the concepts of probability. The most probable equilibrium water temperature expected is computed from the most

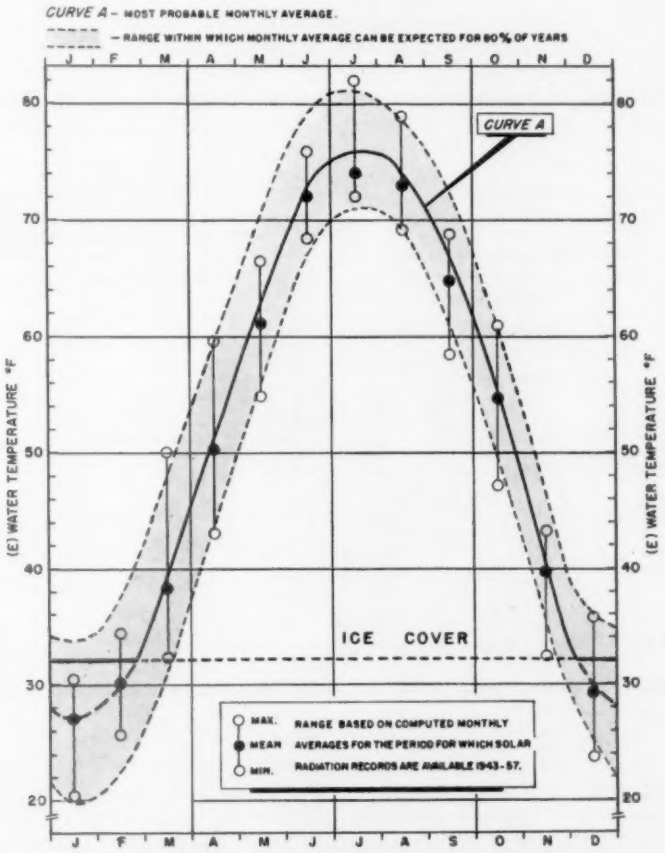

FIGURE 14.—Expected seasonal pattern of monthly average water temperature in lower Tittabawassee Basin, computed from expected seasonal patterns of T_a, W, V_a, and H_a.

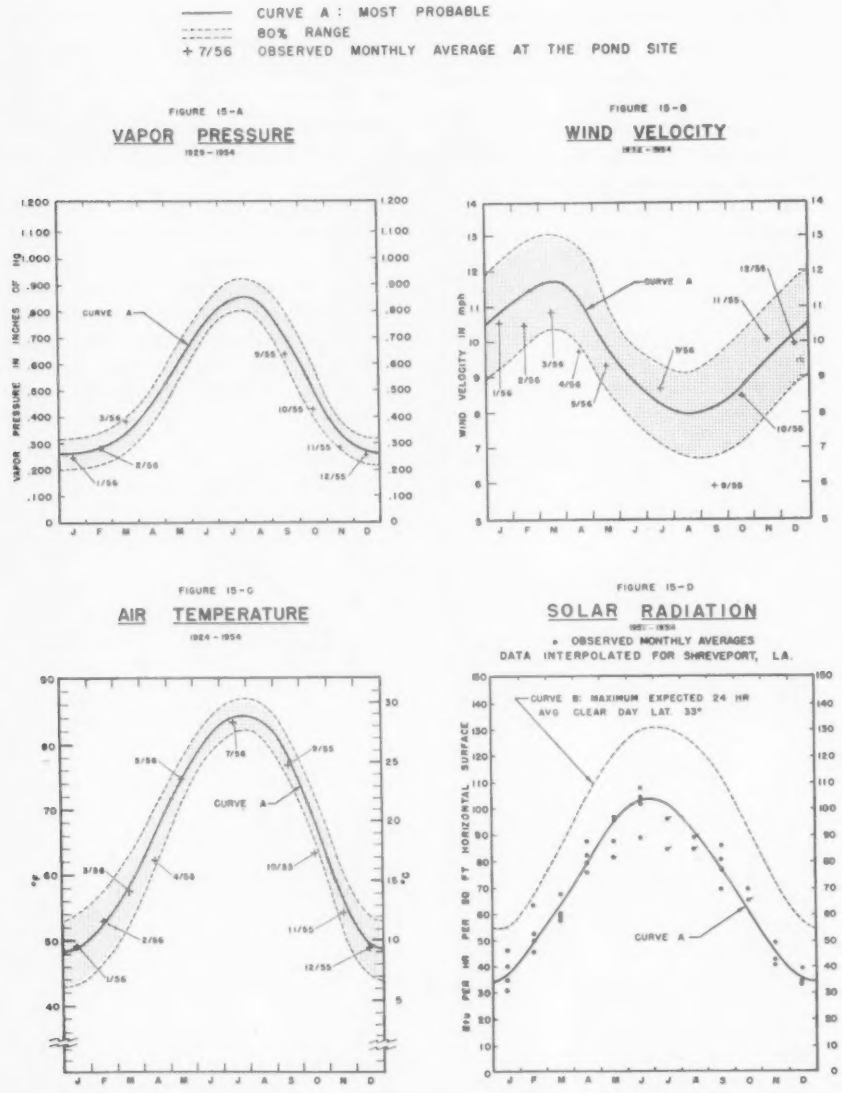

FIGURE 15.—Meteorological seasonal patterns, based on records at Shreveport, La.

probable values of air temperature, wind velocity, vapor pressure, and solar radiation taken from Curves *A* of the respective seasonal patterns. The range within which equilibrium water temperature is expected to fall for 80 per cent of the years is computed for the upper limit on the combination of once in 10-yr high air temperature, vapor pressure, and solar radiation, with the once in 10-yr low wind velocity; and for the lower limit from the combination of once in 10-yr low air temperature, vapor pressure, and solar radiation, with once in 10-yr high wind velocity.

Equilibrium Water Temperature, Vicinity Midland, Mich.

Figure 14 shows the seasonal pattern of equilibrium water temperature as monthly average based on this compromise approach, applied to the Tittabawassee River in the vicinity of Midland, Mich. Curve *A*, the most probable, may be taken as representative of normal, natural, seasonal water temperature expected, and the shaded belt, the 80-per cent confidence range.

As a verification of this approach, individual monthly equilibrium water temperatures were computed for the 14-yr period for which simultaneous meteorologic observations were available at the East Lansing, Mich., Weather Bureau station, 1943–1956. The results of these computations are also shown in Figure 14, the mean designated as solid circles and the maximum and minimum as open circles. There is close agreement between the 14 yearly means and the most probable, Curve *A*; also the minima and maxima of the 14-yr record fall reasonably within the 80-per cent confidence range.

Equilibrium Water Temperature, Vicinity Shreveport, La.

In connection with studies for a wastewater storage and cooling pond, equilibrium water temperatures expected were computed in a similar manner, with heat loss coefficients appropriate to the local site. Figure 15 shows the results of the statistical analyses of the meteorologic records at the Shreveport, La., U. S. Weather Bureau station (located approximately

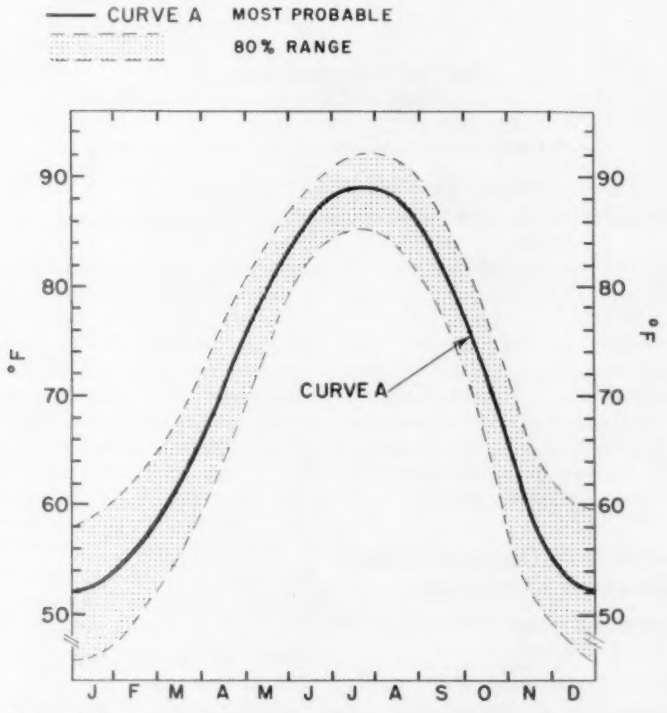

FIGURE 16.—Expected seasonal pattern of equilibrium water temperature at Shreveport, La.

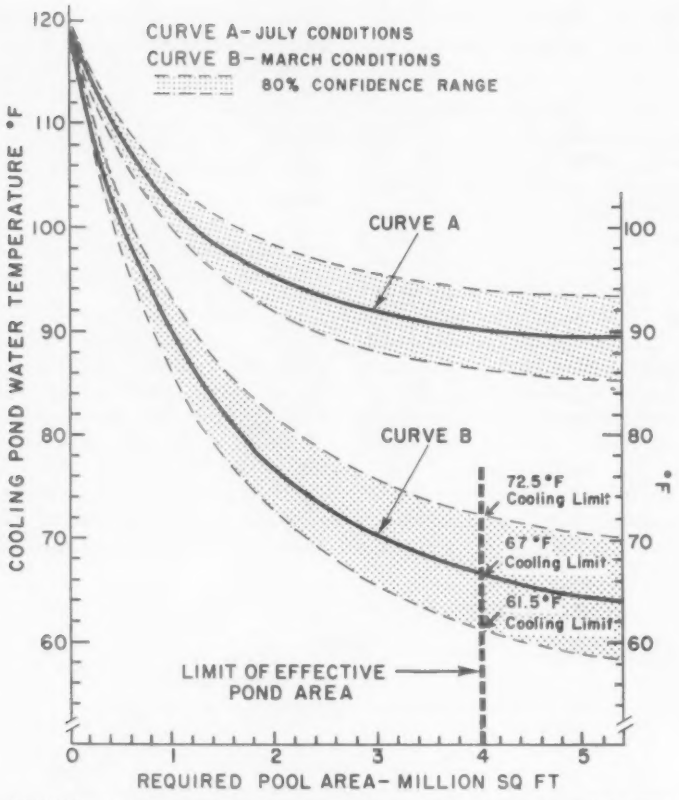

FIGURE 17.—Expected water temperature profile through cooling pond.

40 miles from the basin site), Curves *A*, *B*, *C*, and *D* giving the expected seasonal pattern of vapor pressure, wind velocity, air temperature, and solar radiation, respectively. Figure 16 represents the seasonal pattern of natural equilibrium water temperature expected, computed from the basic meteorologic factors in accordance with the procedure suggested above.

Comparison of Computed and Observed Conditions

Wastewater Storage and Cooling Pond

Following the operation of the wastewater lagoon as a cooling pond, a weather station was erected at the site supplying daily records of three of the meteorologic factors, air temperature, wind velocity, and vapor pressure. Monthly averages of these local measurements are shown in Figure 15 for comparison with the computed expected seasonal patterns based on the official Weather Bureau records at Shreveport. With the exception of the September 1955 wind velocity, the observed data at the pond fit well within the expected seasonal pattern based on Shreveport. With due regard to topographic conditions, it appears that use of long-term meteorologic data at nearby official Weather Bureau stations is warranted.

Also, following operation of the lagoon as a cooling pond, temperatures

of the return water for reuse were taken daily at the return intake, affording a direct comparison with computed cooling temperatures. Figure 17 shows the computed temperature profiles expected through the cooling pond from the inlet of the hot wastewater to the outlet for the return. The wastewater inflow averages 25,000 gpm at 120°F. For the month of July, the height of the warm weather season, Curve *A* represents the most probable temperature profile; the shaded band about Curve *A* represents the 80-per cent confidence range. During this season, with stage of the lagoon high, excess surface area is available and water temperature at the outlet approaches the equilibrium temperature, which for the most probable meteorologic conditions is 89°F and for the 80-per cent range is 85 to 93°F. Curve *B* represents the most probable temperature profile and the 80-per cent range for cool weather conditions in March, a season when the lagoon is at low stage and surface area is limited. For the most probable meteorologic conditions the cooling limit is 67°F, which is 5°F above the equilibrium level. The limiting cooling temperature range expected for 80-per cent of the years is 61.5 to 72.5°F.

From such computed temperature

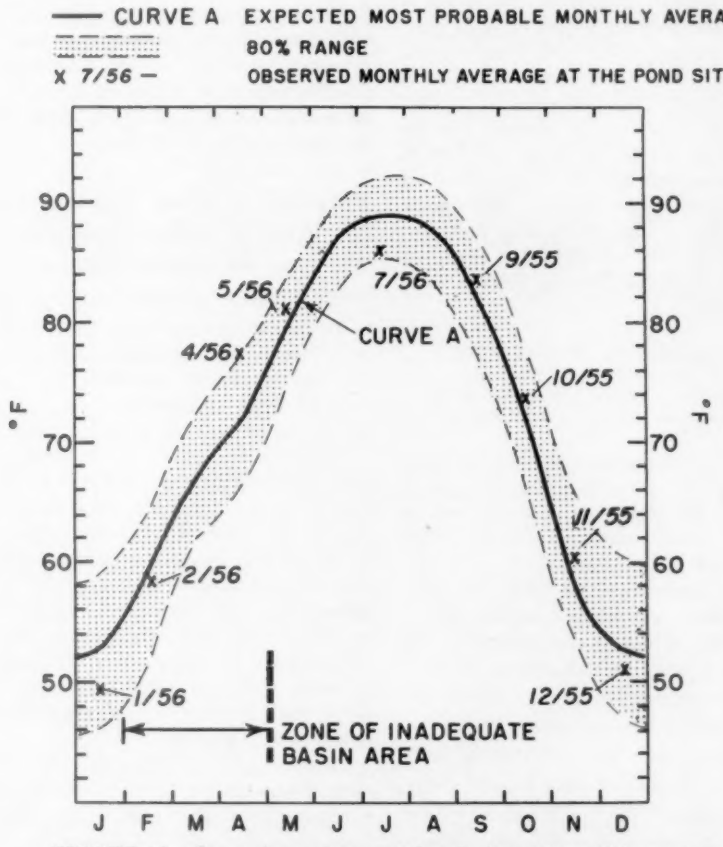

FIGURE 18.—Comparison of computed and observed monthly average temperature at cooling pond outlet.

profiles, Figure 18 was constructed showing the expected seasonal pattern of water temperature of the return wastewater, Curve *A* the most probable and the shaded band, the 80-per cent range. On this graph are also shown the monthly average observed water temperatures of the return water recorded after the pond was placed in operation. They are in close agreement with the computed temperatures.

Tittabawassee River Temperature Profile

For a verification of a computed river temperature profile it is essential that meteorologic and hydrologic conditions remain relatively stable for a reasonable period preceding and during the period of the river temperature survey. Likewise, the introduced heat load inducing the temperature profile should be known and remain reasonably stable for the same period. Such conditions were met for the survey on the Tittabawassee River, September 9–18, 1953. The prevailing meteorologic conditions as reflected at the East Lansing Weather Bureau station were as follows:

Air temperature (T_a)	61.5°F
Wind velocity (W)	10.3 mph
Vapor pressure (V_a)	0.3865 in. Hg
Solar radiation (H_s)	50.8 Btu/sq ft

From these data, the equilibrium water temperature above the introduced heat load, by Equation 8, is computed as 63.7°F. The measured stream runoff (corrected for diversions) at the point of heat load averaged 394 cfs, with only minor increments added along the course. Channel characteristics for this runoff were available from detailed cross-section sounding at 500-ft intervals along the course. Based on these conditions and prevailing heat load of 1.312 billion Btu/hr, the river temperature profile computed by Equation 15 is shown as Curve *A*, Figure 19. Curve *A* shows that the heat load induces a temperature rise from 63.7 to 78.5°F which is followed by a rather rapid heat dissipation with temperature declining to 72.5°F at Mile 19.24 and to 64.5°F at the mouth, approaching again the equilibrium temperature within 0.8°F. The limited observed temperature measurements designated in Figure 19 are in reasonable agreement with the computed temperature profile along the lower reach; likewise, the observed river water temperatures at Mile 24.2 above the point of heat load are in reasonable agreement with the computed equilibrium water temperature, 63.7°F.

Recognizing the sensitivity and dynamic character of the temperature profile, these verifications warrant employment of the basic equations in forecasting river water temperature profiles expected under other natural or regulated stream flows and heat loadings.

Forecast of River Temperature Profiles Under Drought and Regulated Runoff

The critical condition with respect to temperature rise from heat load is coincidence of drought stream flow and meteorologic condition unfavorable to heat dissipation. As an illustration of the influence of drought stream flow and the value of regulated runoff in mitigating heat effect, Figure 20 shows forecasts developed for the Tittabawassee River. As indicated in Table II, severe drought stream flow occurs most frequently in late August or early September. Referring to Figure 14, the equilibrium water temperature for this period as a most probable value is 73°F, and once in 10 yr may be expected to reach 77°F as a monthly average.

Curve *A*, Figure 20, represents the river temperature profile forecast for a combination of meteorologic conditions and drought flow severities expected on the average in the long run

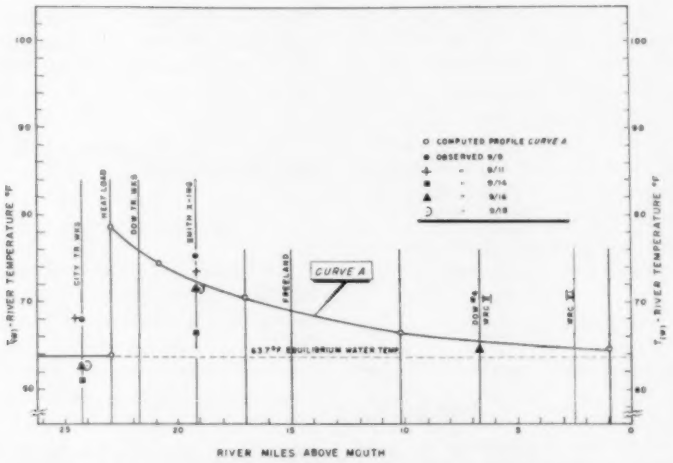

FIGURE 19.—Comparison of computed and observed river temperature profiles below Midland, Mich., on the Tittabawassee River.

once in 10 yr (222 cfs) from a heat load of 1.626 billion Btu/hr. The heat load introduced at Mile 23 raises the river water to 43.1°C. The heat is rapidly dissipated at practically a logarithmic rate, producing a sharp decline in river water temperature. At the low runoff of 222 cfs, time of passage and opportunity for heat loss are longer than at high runoff, practically the entire heat load is dissipated and the water restored to equilibrium temperature in the 23 miles to the mouth of the river.

Curves *B* and *C* of Figure 20 represent river temperature profiles expected under the same conditions as Curve *A*, except that it is assumed the

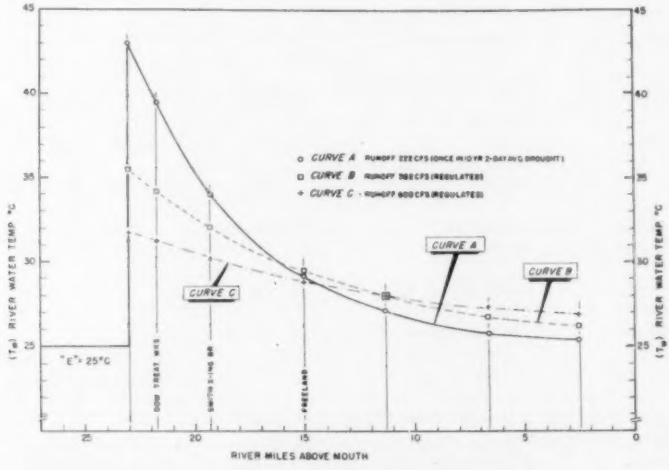

FIGURE 20.—Computed river water temperature profiles below Midland, Mich., in the Tittabawassee River (heat load 4.52×10^5 Btu/sec; $E = 25°C$; $W = 3.8$ mph).

stream flow can be increased by regulation to 382 and 600 cfs, respectively. The effect of the increased stream flow on the temperature profile is pronounced, with much less initial temperature rise and a flatter profile. However, the shorter time of passage at higher stream flow results in raising the profile at the downstream end, requiring a longer reach of river to approach equilibrium temperature.

Conclusions

With a rational method for forecasting expected water temperature conditions, based on probability of occurrence of the controlling meteorologic and hydrologic variables, it is possible to evaluate temperature effects before construction of facilities. This provides an advance in stream sanitation evaluation dealing with an increasing variety of waste heat problems, including site selection for industries and steam power plants. However, it should be cautioned that while there is recourse to mathematical and statistical procedures, these are simply tools, not substitutes for professional judgment.

Acknowledgments

The original basic studies underlying this paper were developed at the School of Public Health, University of Michigan, through stream analysis researches sponsored by the National Council for Stream Improvement of the Pulp, Paper, and Paperboard Industries, Inc. Acknowledgment is made to the Council in supplying data relative to the cooling pond and to the staff of the Michigan Water Resources Commission and to Mr. Thomas J. Powers of Dow Chemical Company for data on the Tittabawassee River.

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MEETINGS OF INTEREST

Apr. 20–22 **Third Biological Waste Treatment Conference,** Manhattan College, Riverdale, N. Y. Professor W. W. Eckenfelder, Manhattan College, New York 71.

Apr. 27–29 **Conference on Algae and Metropolitan Wastes,** Robert A. Taft Sanitary Engineering Center, Cincinnati, Ohio. A. F. Bartsch, Asst. Chief for Special Technical Services, Water Supply and Water Pollution Research, R. A. Taft Sanitary Engineering Center, Cincinnati.

Apr. 29–May 5 **13th International Meeting for the Study of Waters,** Belgian Center of Study and Documentation of Waters, Liege, Belgium. Professor E. Leclerc, 2, Rue A. Stévart, Liege.

May 3–5 **Fifteenth Purdue Industrial Waste Conference,** Purdue University, Lafayette, Ind. Professor Don E. Bloodgood, School of Civil Engineering, Purdue University.

May 6 **Eighth Annual Pennsylvania Clean Streams Conference,** Penn-Harris Hotel, Harrisburg, Pa. R. V. Wall, Pennsylvania State Chamber of Commerce, State Chamber Building, Harrisburg, Pa.

May 15–20 **American Water Works Association,** Americana Hotel, Bal Harbour, Fla. Raymond J. Faust, Secretary, American Water Works Association, 2 Park Ave., New York 16.

May 24–27 **National Rivers and Harbors Conference,** Mayflower Hotel, Washington, D. C. William H. Webb, Executive Vice-President, Suite 523-A, La Salle Bldg., 1028 Connecticut Ave., N. W., Washington 6.

June 5–8 **Seventh Industrial Waste Conference,** Honey Harbour, Ontario. Ontario Water Resources Commission, Parliament Bldgs., Toronto 5, Ont.

June 6–9 **Sewage Treatment Plant Operators Short Course,** Doane College, Crete, Nebr. Division of Sanitation, 1010 State Capitol Bldg., Lincoln 9, Nebr.

June 20–24 **American Society of Civil Engineers,** State Office Bldg., Reno, Nev. W. H. Wisely, Executive Secretary, 33 West 39th St., New York 18.

THE OPERATOR'S CORNER

OPERATING EXPERIENCES AT CAMDEN *

By Vincent P. Brunick

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Camden, N. J., situated on the east bank of the Delaware River directly opposite Philadelphia, Pa., is an industrial city. Wastes contributed by industry are equivalent to the sanitary sewage from a population of over 350,000, or more than two and one-half times the population of the city. About 300 manufacturing concerns, largely devoted to non-metallic products such as food processing, tanning, wool scouring, gas manufacturing, soap making, oil processing, dairies, and dye and chemical works, discharge wastes into the municipal sewerage system. The system consists of 164 miles of combined sewers and 11 miles of force mains and intercepting sewers. The north section of the city is served by the Baldwin Run sewage treatment plant, with a design flow of 3.9 mgd. It receives only a moderate loading of industrial wastes. The great bulk of wastes is treated at the Main sewage treatment plant.

Plant Description

The Main treatment plant provides primary treatment; raw sludge is chemically conditioned, dewatered on vacuum filters, flash dried, and incinerated. Treatment devices, in general, were designed to handle a mean flow of 53 mgd and a maximum flow of 86.7 mgd. All plant conduits were designed to carry a maximum of 105 mgd—the capacity of the intercepting sewer. The 1960 average was estimated to be 46.3 mgd, but the present weekday flow is only about one-half this amount.

Because of the relatively level topography of the municipal area, the invert of the plant influent sewer is approximately 20 ft below ground level. There are two mechanically cleaned trash racks at the entrance to the wet well. Each unit has 3-in. clear openings between bars and each is rated at 53 mgd. Four synchronous motor driven pumps are provided, each rated at 30 mgd at a TDH of 43 ft. Three of the pumps are equipped with variable-speed drives automatically controlled by the liquid level in the pump well. Sewage is pumped from the wet well to two 35-ft square detritors located on the second floor of the service building. After removal of grit, sewage passes through 36-in. diam comminutors to an overhead conduit which leads to the settling tanks. There are eight settling tanks; each is 50 ft wide, 186 ft long, and 10 ft deep. Design detention time is 150 min and the overflow rate is 710 gpd/sq ft. Settled sewage is metered and discharged to the Delaware River. Primary treatment devices are above

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

ground because provisions were incorporated in the design for the future addition of high-rate trickling filters.

Startup

Startup was normal for a sewage treatment plant with inexperienced help, e.g.: Shear-pin breakage was excessive on trash racks, and on sludge and scum collectors; cables were snapped on a trash rack because of faulty replacement of a shear pin; a valve at the sludge storage tanks was opened by mistake during the night shift, returning stored sludge to the main pump well; a third main pump was left on automatic control while only one detritor was in service and during a storm flow the third pump started up before the second detritor was opened, causing the first detritor to overflow; piping in a sludge storage tank collapsed and was replaced by the contractor when investigation disclosed faulty construction; a gear motor on the sedimentation tanks burned because of faulty alignment.

Since the initial breaking-in period, little trouble has been experienced in the primary treatment devices. Operation, nevertheless, has been complicated by the hydraulic underloading of the plant, the types of industrial wastes, and those problems inherent to the physical layout of the plant.

Strength and Quantity of Sewage

When the treatment plant was placed in operation, approximately one-third of the sewage of the city was, and still is, discharged untreated. On the other hand, the wastewater reaching the plant contains most of the city's industrial wastes. As a result, the sewage treated is high in suspended solids and BOD. Another factor which has an effect on the strength of the sewage is the capital improvement program instituted by the water department. Only about 22 per cent of the properties were metered for water at the time the plant began operation. At nearly the same date, though, the city started a program to meter all properties and at the same time increased the water rates. Water users, faced with increased water bills, conserved water.

Since a large portion of the flow is from industry, the peak loads are from about 9:00 AM to 6:00 PM, Monday through Friday. During week ends the dry weather flow sometimes makes it necessary to raise the liquid level in the pump well by surcharging the intercepting sewer, and to pump at minimum speed on manual control to prevent frequent stop-start operation of the 300-hp synchronous motor with its variable speed drive. On the other hand, a rain storm can cause the rate of flow to change from 15 to 86 mgd (the present settings of the regulators) in 90 min or less. When this happens, a large quantity of grit and screenings reach the plant taxing the trash racks, detritors, and grit ejectors.

The first effect of plant underloading was felt in the main pump well and pumps. Settling of solids in the pump well made it necessary to back-flush the suctions of standby pumps before putting them in service during storm flows. After nine months of operation, the pump well was cleaned and a program was begun to pump down each well and back-flush each pump twice weekly.

From the pump well the sewage is lifted to the detritors. Here, because of the low flow and the relatively high specific gravity of the cannery wastes, the grit and large quantities of California pea beans, corn, carrots, potatoes, onion skins, and tomato seeds and pulp settle out. On an annual basis, this so-called grit amounts to 6.5 cu ft/mil gal of sewage treated, but it reaches the plant in shock loads of up to 240 cu ft/day during storm flows and during the twice-weekly pump-downs of the wet well.

Of the eight settling tanks provided, only three or four are used, depending on the flow. The concentration of suspended solids in the plant influent is over 400 mg/l; the average removal is 75 per cent. Weekly flushing of the inlet channel is necessary to prevent deposits, even though it is aerated. Preservation of equipment in the unused tanks has been a major task for the maintenance force.

Sludge Handling

While most of the difficulties in the sewage treatment section of the plant have been due to underloading, those problems in the sludge disposal section of the plant have been caused by too much sludge—or rather to the inability to get rid of the accumulated sludge.

The two sludge storage tanks are equipped with floating covers, supernatant take-offs, and the heating and control devices usually found in standard digestion tanks. Each tank is 70 ft in diameter with a side wall depth of 26 ft making the tank capacity 100,000 cu ft. At design flow, they provide storage for about six days, or they could provide digestion for about 20 per cent of the sludge if it would seem advisable to filter a mixture of raw and digested sludge.

From the storage tanks, sludge is pumped to a sludge well at the sludge disposal building. From the well, it is lifted by three bucket elevators to three mix tanks where ferric chloride and slaked lime are added. The speeds of the bucket elevators, ferric chloride pumps, and lime slakers are controlled by the sludge level in the mix tanks. From the mix tanks, conditioned sludge flows by gravity to any of six 8-ft by 10-ft vacuum filters. Filter cake is routed by conveyor belts to either one or the other or both incinerators. Each incinerator is designed to flash dry and burn filter cake (having a moisture content of 70 per cent and a heat value of 5,200 Btu/lb) at a rate of 10,000 lb/hr.

Contract specifications provided that both the filter and the incinerator manufacturer furnish 30 days' instruction to operating personnel. During this time the manufacturers also were to make the necessary adjustments to their equipment. Immediately following the training period, a 30-day acceptance test was to be run on each of the incinerator units. But it was not until 5 months later that the first tests were attempted and not until 14 months after startup that the tests were completed. During this period it was difficult to maintain a steady flow of filter cake to the incinerators. Contrariwise, during good filter runs it was often necessary to stop for awhile to make adjustments to the incinerators. Stratification in both the sludge storage tanks and the sludge well of the filter produced at times sludge either too thick or too thin to filter easily.

Dewatering and Incineration

For a prolonged period, the concentration of hair in the sludge caused the cake to come off the filter drums in the form of a "felt" mat, $\frac{5}{8}$ to $\frac{7}{8}$ in. thick and 8 ft wide. This mat folded over itself several times on the conveyor belt before it broke loose from the filter. The filter-cake breaker could not smooth out belt loadings such as these. The load on the incinerators fluctuated from a 25-per cent load to a 100-per cent overload in a matter of minutes. The hair, when it entered the flash drying system, had a tendency to remain suspended in the cyclone until enough would accumulate to plug the air-lock causing an emergency shutdown. Also during this period, material in the sludge was causing excessive wear to equipment, dangerous buildups of grease in the drying system, and frequent plugging of the cage mill.

A great deal of time, labor, and money were expended with varying degrees of success before investigation

showed that the sludge, accepted as normal for the plant, was in fact abnormal. The three main causes were: (*a*) the holding of the sludge for months until a sufficient amount was accumulated to start filtration and incineration; (*b*) the shock loads of various industrial wastes which reached the plant during startup as main sewers were diverted to the plant one by one; and (*c*) the seasonal variation in the industrial wastes reaching the plant.

A determined effort was made to get rid of this type of sludge, regardless of operating efficiencies. If four or five filters were needed to load an incinerator they were used. If an incinerator balanced out at 60- or 70-per cent load, it was operated at this load. As the characteristics of the sludge changed, emergency shutdowns became less frequent and of shorter duration, and filter runs were increased from 8 to 12 hr and finally, in the fall of 1957 (about 17 mo after startup), to two full shifts per day.

Sludge Storage

Raw sludge is pumped from the settling tanks to the sludge storage tanks during the day and afternoon shifts. The next morning, about 5:00 AM, the midnight shift starts to remove the supernatant, and this continues on the day shift until solids appear at the sampling manhole.

Experience has indicated that digester-type tanks are not ideal for storing raw sludge. It was evidently presumed that the raw sludge would settle and concentrate at the bottom of the tanks; instead, the sludge stratifies on standing. Decanting of supernatant is usually difficult and sometimes impossible. Bottom sludge is often too watery to filter and too heavy to return to the plant influent and must be pumped to the other storage tank until a heavier layer reaches the bottom. Many times the sludge at a supernatant withdrawal line is ideal to send to the filters, but this cannot be done because of the present piping arrangement. Recirculation and tipping of tank contents do little except rearrange the sludge layers. Changes in the piping could eliminate many of these headaches, but, at the present time, pumping-station and sewer projects have priority over this needed major alteration.

Tanks have been operated in series and in parallel. For the last six months of 1958 and the first three months of 1959, sludge was pumped to sludge storage tank No. 1 through the high inlet. All possible supernatant was decanted and sludge was transferred from the bottom of tank No. 1 to tank No. 2. Additional supernatant was removed and the sludge was then pumped to the filter well. This arrangement worked fine until the solids in the sludge slowly and steadily decreased. An inventory of tank contents revealed a buildup of sludge averaging 11-per cent solids in the upper third of tank No. 1. At the present time, sludge is pumped to one tank for seven days while sludge in the other tank is sent to the filter sludge well. Sludge is pumped to the filter sludge well every three hours during filter runs because it will stratify again if allowed to stand for too long a period.

Filter Yields

The major item affecting sludge disposal at Camden is the characteristics of the sludge. This item is followed in importance by solids concentration and chemical conditioning. Filtration rates are highest during the tomato canning season, when filtration rates of from 4.2 to 5.0 psf/hr (dry solids) are obtained; the yearly average is 2.8 psf/hr. Moisture in the filter cake drops from a yearly average of 71 per cent to about 66 or 67 per cent, and solids in the sludge increase from a yearly average of 6.5 per cent to 9 per cent or more. The heat value of the

cake increases from an average of 3,400 Btu/lb to a high of 5,000. For this reason, the incinerator's load is determined not by the weight of the filter cake but by the temperatures in the critical areas.

On the opposite end of the scale is that sludge encountered, usually after storm flows, which, regardless of the per cent of solids, produces a very thin cake which is low in heat value and quickly blinds cloths.

Slugs of hair which accumulate in the storage tanks cause the cake to be thick and easy to filter, but the dewatered cake has a tendency to crack during the drying cycle and as a result its moisture content approaches 75 per cent.

Best filtration yields are obtained with a sludge having about 8 per cent solids. As the solids content approaches 10 per cent, plugging of the lines between the mix tanks and filter troughs occurs. Solids concentrations below 6 per cent produce a very thin cake and require more chemicals for conditioning.

Control Test

As previously stated, speeds of the bucket elevators, ferric chloride pumps, and lime slakers are controlled by the levels in the chemical mix tanks; the levels in the mix tanks are determined by the number of elevators and filters in operation. Thus the detention time in the mix tanks also varies. These variables all affect chemical conditioning to some degree.

During startup and for many months afterward, chemical conditioning of the sludge was a major problem and operational control was poor. Ferric chloride and lime dosages were calculated using Genter's formulas (1). Data, however, could not be obtained fast enough from the laboratory when sludge concentrations were changing rapidly or when one or more filters were failing. Needed was a relatively simple test for conditioned sludge which could be run hourly by the filter operator to find out whether more or less ferric chloride or lime was needed, whether a ferric chloride feeder or lime slaker was metering as set, or whether a blanket was blinding. In other words, a control test was sought which would give the operator advance warning of an impending drop in the filtration rate.

Buchner funnel tests were run on the conditioned sludge during good, bad, and indifferent filter runs and it was discovered that during good filter runs the funnel break-time and the pH of the funnel filtrate were within certain definite limits. As a result, the following test procedure was set up at the filter platform: 200 ml of the conditioned sludge is poured into a 90-ml Buchner funnel and a closed-system vacuum of 20 in. mercury is applied. For suitable dewatering, the vacuum break-time will be between 90 and 120 sec and the pH will be about 11. This test is run hourly on mix tanks in service and if, during a filter run, the break-time increases and the pH decreases, the lime slaker is immediately checked for proper metering. If the pH is high, the ferric chloride pump is checked. Many times this test has allowed corrective action to be taken before blinding of the cloths has occurred.

Conclusions

Although operating problems caused by the industrial wastes have been many, they are being reduced gradually. Some operating procedures once regarded as unusual are now accepted as routine; others, such as the improvement in handling of stored sludge, must await alterations in existing structures.

The completion of construction of interceptor sewers with the resulting increase of flow of mostly sanitary sewage is expected to further elimi-

nate some of the troubles now encountered.

Acknowledgment

Appreciation is expressed to John Frazee, Superintendent of Sewage, for his advice and assistance in preparing this paper, and to William A. Turner, Jr., Chemist for the Bureau of Sewage.

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CURVED SEWERS AT BURBANK *

By Ervin Spindel

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The practice of using curved alignment for sanitary sewers has been found quite acceptable by many agencies in Southern California.

The primary purpose of curved sewer alignment from an engineering standpoint is economy of construction. It cuts down the length of sewer and the number of manholes. For example, for a 90-deg bend which ordinarily would be made at a manhole, a 100-ft radius curve would save 43 ft of pipe, and possibly the manhole.

A secondary purpose, especially on the large sewers, is to improve hydraulic characteristics and eliminate turbulence at manholes.

There are no apparent difficulties in the maintenance of curved sewers. With modern cleaning equipment they can be easily cleaned. Maintenance men, with whom this matter has been discussed, have no objection to curved sewers if proper criteria of design and construction are adhered to. The practice of looking through the sewers to see if they are obstructed is not really necessary. Determination of whether or not there is an obstruction between two manholes usually can be made by observation of the relative depths in the manholes. If the depth of sewage in the upstream manhole is appreciably greater than that in the downstream manhole, there is generally an obstruction in the sewer.

The radius of curvature should be no less than 100 ft. On sewers 15 in. in diameter or smaller, square-end pipe can be used on a 100-ft radius without excessive pulling of joints. On pipe larger than 15 in., beveled pipe must be used in order to obtain good joints on a 100-ft radius; if beveled pipe is not used, the radius should be increased sufficiently to keep the joint opening within acceptable limits. Pipe joints should be made carefully and inspected with particular attention paid to the pointing and wiping of the inside joints to make sure no mortar fins or irregularities are left in the pipe.

There should be no more than one curve, either horizontal or vertical, between any two manholes. Manholes should be placed, if possible, at the beginning and end of the curve. If these criteria are followed, there should be no trouble in maintaining curved sewers.

The city of Burbank has in use about 8,000 ft of curved sewers in 30 separate installations. The grades of these sewers vary from very flat to very steep, and the diameters vary from 8 to 30 in. During the approximately 5½ yr that curved sewers have been used, there has been not one stoppage nor any unusual maintenance difficulty caused by the curves.

* Presented at the 31st Annual Conference, California Sewage and Industrial Wastes Assn.; Long Beach, Calif.; Apr. 29–May 2, 1959.

AIR POLLUTION CONTROL IN SEWAGE TREATMENT PLANTS *

By Rolf Eliassen and Clement A. Vath

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Air pollution control has become a problem of increasing importance to many sewage treatment plant operators as metropolitan areas have expanded and virtually surrounded plants which were originally in isolated areas. Newspapers and popular magazines have been stressing the detrimental effects of air pollution on health. Public awareness of air pollution has led to a growing intolerance of the paradox of cleaning up the streams at the expense of polluting the atmosphere.

The Problem

Odors constitute the principal air contamination problem at many sewage treatment plants. Sewage contains microbiological organisms, organic foods, and mineral nutrients. This combination is used to advantage in treatment plants utilizing biological processes such as trickling filters, activated sludge, and digesters. These are controlled biochemical reactions which should not lead to significant odors under efficient plant operation, provided no odorous industrial wastes are present.

But biological degradation can take place in the sewer systems and at the head end of treatment plants where oxidizing agents such as air or chlorine are not added. Exhaustion of dissolved oxygen in sewers leads to anaerobic fermentation in the acid stage to produce such obnoxious products as indole, skatole, cadavarine, mercaptans, amines, and sulfides. In a digester these are broken down under controlled conditions. Any odorous gases are collected and burned—oxidized to innocuous end-products. Long, flat sewers flowing full, or nearly so, carrying strong sewage in regions with high temperatures contribute the greatest problems, particularly where open manholes exist, or at the sewage pumping station where the anaerobic sewage is first exposed to the air.

Many other conditions can give rise to odors beside the odor of incoming sewage. In manually cleaned grit chambers, accumulation of excess grit containing organic matter is a common source of odors, as is the exposure of collected grit, screenings, or grease. Anaerobic conditions can arise in any unit of a treatment plant if not properly designed or operated.

Sludge is also a source of odors, particularly if drawn to drying beds before digestion is complete. These odors are difficult to control unless drying is fast and rainfall does not occur. The earthy, tarry odor of well-digested sludge is not generally considered to be a problem.

Incineration or mechanical drying of sludge has led to many an odor complaint, particularly the pungent odor of decomposed proteins when proper operating temperatures are not maintained in the drying apparatus. Dust is also a problem with some sludge drying equipment.

Corrective Measures

Once the source of pollution has been located, two avenues of approach are available for the elimination or reduction of the problem. The source of pollution can either be prevented from occurring or its emission can be con-

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

trolled before it enters the atmosphere. Prevention entails the remedying of the condition which initially is the cause of the problem, whereas control seeks to reduce the amount of annoying substances reaching the atmosphere. The method of attack employed depends largely on the nature of the problem, its severity, and economics (1)(2)(3)(4)(5)(6)(7)(8).

The fundamentals of prevention are adequate, skillful, and thoughtful plant design, and continual good housekeeping once the plant has been put into operation. Where odors are the consequence of bacterial degradation of organic matter present in the sewage, prevention consists of modifying or terminating the degradation process. To these ends, oxidants such as chlorine, nitrates, or oxygen have been added to supply a chemical oxygen source to supplant the use of sulfates and organic sulfur compounds for the oxygen needed for stabilization. Iron salts have been employed to precipitate the obnoxious sulfur compounds from solution, although rather inefficiently. Bactericidal agents like chlorine and bacteriostatic agents like orthodichlorobenzene have been used in sewers to suppress troublesome areas. Tall oils and water sprays have been found effective in controlling foam on aeration units and in so doing prevent the deposition of odor-producing grease along tank walls and walkways. Furthermore, many potential odor sources can be eliminated by keeping a clean, attractive plant. It has been appropriately stated that 90 per cent of odor problems originate with the eye rather than the nose.

With pump stations, screens, and grit chambers receiving septic or odorous sewage it may be advisable to enclose these units, install an induced draft fan to keep the housing under a negative pressure, and then oxidize the odorous gases. Absorbers such as activated carbon or silica gel could be used to remove the organics from the air stream. Oxidation by passing the gases through a furnace or catalytic combustion unit or oxidizing with ozone or hydrogen peroxide could also be utilized.

Covering of settling tanks and oxidizing the exhaust gases have been practiced at many plants, notably at the Owls' Head plant in New York City. Ozone is added to all of the gases drawn from the top of the settling tanks and other covered parts of the plant. This procedure had to be adopted in order to obtain approval for the use of this recreational area adjacent to good housing. Sarasota, Fla., has employed a plastic covering over a trickling filter. Many filters have concrete domes which can be vented through an oxidizing medium. Even when aerobic, sewage does have a characteristic odor considered objectionable by many people. More covered units will surely be in evidence in future designs.

Odor modification may be a masking or counter-odor process. Many masking agents have been used in the authors' experience, including odors of pine woods, the streets of Paris, and some aromatics, as well as some of the more sophisticated and highly useful products now on the market. The effectiveness of masking is based on the principle that when two odors are mixed, the stronger one will predominate. It is hoped that the stronger one will be pleasant to all people. Odor counteractants are also being used, with varying degrees of success, depending so much on the relative reactions of the olfactory nerves of the public. Attempts have even been made to spray substances into the air to deaden olfactory responses. Formaldehyde will do this, but it is not a safe public health practice. It is far better engineering practice to control the odors at their source, or to collect odorous gases and oxidize or absorb them from the air stream.

Dust control is a highly developed

art, widely practiced by all types of industries. Many sludge drying plants use cyclones as the sole method of dust control. These will remove only coarser solids (above 10 microns). Scrubbers can remove sludge particles below a micron in size, while electrostatic precipitators, as used in many power plants, will take out virtually all dust down to 0.01 micron. Sludge dryers and incinerators have been designed to operate successfully with air cleaning devices to cut down smoke and dust emissions.

A Look at the Future

Design engineers will be called on to provide more facilities for air pollution control at sewage treatment plants—both new and old. These facilities will include covering and ventilating many of the structures in which sewage would otherwise be exposed to the atmosphere. More advanced air cleaning techniques will be provided the plant operator to control odorous gases—such devices as gas absorbers, adsorption beds, chemical or catalytic oxidation and combustion units—to make possible the efficient treatment of the collected gases. Up-sewer chemical feeding stations for iron salts, chlorine, or chlorinated hydrocarbons have been and will continue to be included in the design of long interceptor sewers in hot, flat country to reduce the odor of the sewage reaching the plant. Odorous industrial wastes or combinations of wastes which will create odors in sewers will be separated and treated at the industrial plants.

More attention will be paid by designers to sludge drying beds in plants near populated areas. It will not be necessary to use glass greenhouses to control the odors. Perhaps a new plastic covering, or other synthetic material can be designed to cover the bed while heated air flows over the surface of the wet sludge in the bed to accomplish fast drying. The moist air containing any odorous gases could pass through a combustion unit for odor control. Then the plastic cover could be removed for sludge withdrawal and the operation repeated.

This would make sludge beds more expensive—air pollution control costs money—and other means of sludge disposal may become more attractive economically. Vacuum filtration, sludge flotation by the Laboon process, and other methods yet to be developed, either with or without sludge drying or incineration, will all play a part in solving the air pollution problem for operators. More sophisticated dust control equipment will be employed in many plants, particularly air scrubbing with plant effluent, multicelones, filters, and electrostatic precipitators. There is a considerable amount of research yet to be done on improved methods of odor control in sewers and sewage treatment plants.

With such well-engineered plants the operator will be in a better position to do a thorough job. As in the treatment of liquid wastes, the efficiency of the plant depends upon the diligence of the operator. For awhile he may be able to camouflage plant odors by strong masking agents or odor counteractants, but there is no substitute for sound engineering for air pollution control and intelligent operation. Then the operator will truly be in charge of an entire wastes disposal system handling the air-borne, liquid, and solid wastes in such a manner that the resources of the environment—air, water, and land—are protected and preserved for the best use of the public.

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MAINTENANCE RECORDS FOR SEWERS AND LIFT STATIONS*

By Ernest Hamilton

Superintendent of Sewers and Sewage Treatment, Boulder, Colo.

Maintenance records for lift stations and sewer mains are a necessity. Records can and should be clear and concise and, in most cases, quite simple. Usually the time needed to keep good records will not be more than 10 or 15 min a day. The old idea that a foreman or superintendent can remember the work that has been done should be rejected for all time.

Sewer Main

In Boulder, records are held to a minimum. Although, the time spent on keeping them is a little longer than hoped for, every minute is considered to be well spent. The rodding and bucket machine crews use a blank street-type record sheet on which is shown a diagram of the work area. Supplied in this diagram are the distances between manholes, depth of manholes, condition of manholes and main, and any unusual conditions surrounding the area, such as large trees, low areas, and culverts. These sheets are handed to the foreman at the end of the day. The foreman then makes out a daily work sheet (Figure 1) listing the information from the sketch sheets turned in by the crews. Items on the sheet cover the area or location (street and alley), employees on the job, man-hours, and the account numbers to which the work should be charged. Similar information is recorded for the equipment and materials used.

Weekly, the superintendent and the foreman fill in permanent record cards on the sewer system which show: (*a*) the location, (*b*) the manhole numbers, (*c*) pipe size, (*d*) manhole depths, (*e*) distance between manholes, (*f*) their condition, (*g*) amount of infiltration, if any, and (*h*) condition of line at the time of maintenance. Listed on the back of these cards are the: (*a*) date of emergency call, (*b*) time of call, (*c*) time crew reported to call, (*d*) condition of line on arrival, (*e*) emergency remedy, and (*f*) permanent remedy recommended.

Sewers cleaned in response to emergency calls are noted on the sewer map, and after three pins show up in the same section of main in one year, the main is repaired or replaced or trees, etc., are removed to remedy the problem. Narrow, colored tape is used on a sewer map for each year's progress in cleaning. This gives all concerned a quick picture of the status of the maintenance work. Routine sewer cleaning is on a two-year cycle

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

CITY OF BOULDER, COLORADO

Sewer Maintenance Department Date_____

Location:

Men Used	Hours	Account No.	Material Used
Equipment Used	**Hours**	**Account No.**	
			Distance between manholes_____
			Depth of manhole #_____
			Depth of manhole #_____
			Distance between manholes_____
			Depth of manhole #_____
			Depth of manhole #_____
			Distance between manholes_____
			Depth of manhole #_____
			Depth of manhole #_____

GENERAL REMARKS:_____

and trouble spots, twice a year. This program has cut emergency calls by more than 80 per cent in the past three years.

A monthly report is prepared and submitted to the Director of Public Works by the Superintendent of Sewers. This report is then combined with monthly reports from other department heads and the over-all report is submitted to the City Council through the City Manager. At the end of the year all the monthly reports are consolidated into an annual report. These monthly and annual reports are very helpful in preparing budgets. Boulder uses a performance-type budget and the information from all records makes preperation and defense of budget requests much easier.

Lift Stations

At the present time two lift stations are in operation and two more are planned for the near future. Operation is handled by the operators at the treatment plant as one of their duties. The records for the present stations consist of a notebook with sheets carrying the following headings: (*a*) kw-hr used and cost, (*b*) greasing schedule, (*c*) motors, (*d*) pumps, (*e*) flow, and

(*f*) buildings and grounds. Power consumption and cost are self explanatory. Under greasing schedule, the dates of the last greasing and oil changes are shown, as well as the date of the next greasing and the type and quantities of oil and grease to be used. The motor and pump sheets show all the name plate data, dates of any repair, type of repair and maintenance, and the cost of same. Maintenance of buildings and grounds can become a large maintenance item, but paint and repairs are a must.

Summary

Records are necessary, but they need not be complicated. The extent of record keeping will be determined mostly by what is wanted from records and the dictates of the community. Records are valuable in planning programs, defending law suits, and justifying budget requests. They supply fundamental data which, when properly interpreted, form bases for decisions on equipment replacement and major repairs.

TREATMENT PROBLEMS CAUSED BY CUCUMBER CANNING WASTES *

By Floyd L. Kimball †

Superintendent, Sewage Treatment Plant, Crosswell, Mich.

The sewage treatment plant was put in operation late in October 1957. The plant has a grit chamber, Imhoff tank with 4 settling compartments, high-rate trickling filter with recirculation, final clarifier, and chlorinator.

The plant was designed for a BOD loading of 1,440 lb/day, and the waste from the pickle canning industry was not to exceed 444 lb/day. The pickle waste is acid and its pH runs from 4.0 to 6.0 most of the time. The plant influent was slightly on the acid side and usually had a pH of from 6.0 to 6.5.

The plant was put into service at the start of cold weather, and for this reason biological action in the digestion compartments was probably slow in developing. In the beginning of May 1958, the settling compartments started to turn black. This change continued as the weather and sewage became warmer; it was caused by the activity in the digestion compartment. Partially digested sludge was rising to the flow-through chambers and passing on to the trickling filter making a strong filter influent. Sometimes this influent had a much higher BOD than the plant influent.

Loading

Overload

An examination of Table I shows the BOD load in the raw waste averaged 2,144 lb/day when it should not have exceeded 1,440; the pickle company waste BOD load averaged 1,277 lb/day and it should not have exceeded 444. The design load for the filter was 936 lb/day (31 lb/1,000 cu ft), but the applied load was 2,144 lb/day (69 lb /1,000 cu ft), the same as the influent. At these high loadings, filter efficiency was 56 per cent; the removal averaged 38 lb/1,000 cu ft.

Some volatile acids determinations were made on digesting sludge and they were high, but within common operating limits. The volatile content of total solids was about 40 to 45 per cent; the pH was 6.5.

With this overload there were odor

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† At present, Superintendent, Sewage Treatment Plant, Lapeer, Mich.

TABLE I.—Trickling Filter Performance during Overload

Date (1958)	Flow (mgd)	Raw Waste BOD (mg/l)	Pickle Waste BOD (mg/l)	BOD Applied to Filter (lb/1,000 cu ft)	BOD Removed by Filter (lb/1,000 cu ft)	Eff. (%)
7/23	0.312	2,815	1,110	61	39	64
24	0.326	2,900	1,720	57	33	58
25	0.348	4,030	1,660	72	40	56
28	0.665	1,680	1,120	36	17	47
29	0.600	1,515	760	44	33	74
8/ 4	0.604	2,665	1,560	61	43	70
5*	0.765	2,560	1,890	128	54	42
12*	0.925	1,930	1,472	129	69	54
13	0.749	2,124	1,450	46	15	33
14*	0.762	1,520	1,132	78	39	50
15*	0.768	2,040	1,410	74	39	53
19*	0.760	1,395	470	60	23	38
22*	0.785	1,310	1,060	67	43	64
26*	0.800	1,540	1,060	54	43	78
Avg	0.655	2,144	1,277	69	38	56

* Days of excessive digestion activity; Imhoff tank effluent strength exceeded influent strength.

and corrosion problems and the growth on the trickling filter was not healthy.

Normal Load

During Sept. and Oct., 1958, the seasonal slowdown at the pickle plant greatly reduced the load on the treatment plant. Data in Table II show the job that can be done when the plant operates at a normal load. With an average BOD load of 18.7 lb/1,000 cu ft, the removal was 15.6 lb/1,000 cu ft or 85 per cent.

TABLE II.—Trickling Filter Performance during Normal Load

Oct. 1958	BOD Applied to Filter (lb/1,000 cu ft)	BOD Removed by Filter (lb/1,000 cu ft)	Eff. (%)
1	9.6	8.9	93
2	15.6	13.9	90
3	16.6	14.8	89
6	9.2	8.6	93
7	10.9	10.1	92
8	19.1	17.1	90
9	10.3	7.8	75
10	16.3	14.4	89
13	26.2	22.2	85
14	31.3	21.0	68
15	7.9	6.9	87
16	21.8	18.9	87
17	25.5	19.5	76
20	7.5	7.0	93
21	10.4	9.6	92
22	25.8	23.6	92
24	20.6	17.4	84
27	31.7	28.6	90
Avg	18.7	15.6	85

These data come from the results of the examination of 9-hr samples and for this reason the results are probably higher than those which would obtain with 24-hr samples.

Discussion

Probably the main reason for the plant overload was the increase in the demand for sweet pickles and sweet relish which took place between the time of the original waste analysis and the time the plant was completed. Sugar was the big contributor to the high BOD.

City officials and health department and pickle company personnel are actively working on the waste problem.

The following monthly sewer service charges were in effect for the pickle waste:

30¢/1,000 gal for the first 50,000 gal
20¢/1,000 gal for the second 50,000 gal
12¢/1,000 gal for the next 900,000 gal
4¢/1,000 gal for the next 1,000,000 gal
2¢/1,000 gal for all over 2,000,000 gal

Thought is being given to increasing the last two rates.

PERSONNEL, TOOLS, AND EQUIPMENT FOR SEWER MAINTENANCE *

By A. E. Holcomb

Assistant Sanitary Sewer Supervisor, Dallas, Tex.

The first sanitary sewers were constructed in Dallas between 1870 and 1880, with the sewage being discharged into the Trinity River untreated. In 1906, the first sewage treatment plant was built. At present, there are approximately 2,000 miles of sanitary sewer mains and 169,000 house connections in the collection system. The sewer mains vary from 4 to 90 in. in diameter and cover an area of over 273 square miles. These sewers serve some 690,000 people. About 75 per cent of the sewer mains are in the 6- and 8-in. pipe size group and these are the sewers which require the most maintenance. About 50 per cent of them have been built in the past 20 years which is quite an advantage as far as sewer maintenance is concerned. building connections from the main to In Dallas, all the sewer mains and all the property line are maintained by the city. The sewer system handles only domestic sewage and industrial wastes.

Personnel

The Sanitary Sewer Maintenance Division in Dallas is a part of the Water Department and is directly under the supervision of the water superintendent and the assistant superintendent. Its duties are to maintain the collection system and install sewer laterals for new buildings (Figure 1). The division does not handle new sewer construction or sewage treatment; these are handled by separate divisions.

There are approximately 210 people employed in the maintenance division and 10 of these work in the office. The others are included in the following groups: (*a*) 4 emergency crews (Figure 2); (*b*) 27 repair and new service crews; and (*c*) 7 sewer-cleaning crews. The city is divided into four districts with each district having its own personnel, buildings, and equipment necessary for maintenance work. In each district there is a general foreman who is in charge of all work except for sewer cleaning and emergency work. These two phases of the work are under the supervision of another general foreman who works the entire city with these crews. All of the field work is under the supervision of a field supervisor who receives his orders directly from the sanitary sewer supervisor and the assistant supervisor. The Division is fortunate in having some of the best men in the sewer maintenance business as general foremen. These men have all come up through the ranks and are definitely well qualified for the positions they hold. There are also some very competent employees in the laboring group also.

Tools for Sewer Maintenance

Pipe Cutters

The pipe cutters † used will cut cast iron soil pipe up to 6 in. in diameter and tile and concrete pipe up to 15 in. in diameter. The cutter consists of two metal handles and a chain which is fitted with small metal discs spaced at close intervals. One end of the chain is connected permanently to one handle and the other end hangs loose. The chain is wrapped around the pipe with the handles spread apart. The loose end of the chain is hooked snug into a

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

† Manufactured by the Wheeler Manufacturing Corp.

FIGURE 1.—Specially designed truck used in sewer repairs and new lateral construction.

notch in the other handle. This lets the two handles spread apart at about a 45 deg angle. Pressure is put on the handles by pushing them together and this causes the chain to tighten and put uniform pressure on each of the discs around the barrel of the pipe. The pressure from the discs causes the pipe to break uniformly along the chain line.

FIGURE 2.—Specially designed truck used in emergency work.

Dipping Needles and Pipe Locators

There are several different types and brands of dipping needles and pipe locators which can be purchased at reasonable cost. It has been found that much time and labor can be saved with these two instruments. The dipping needles are magnetized to pick up cast iron and steel and are very helpful in locating lost manholes and cleanouts covered with dirt or pavement. In using the pipe locators, a steel cable is pushed into the sewer that needs to be located and then the locator is connected to the cable. The cable, then, can be traced as easily as any underground metal line.

Safety Equipment

Gas Masks

The gas masks used are Chemox oxygen-breathing apparatuses.* The mask is equipped with a canister which supplies oxygen to the wearer. As the wearer breathes into the canister the moisture from the exhaled breath causes a chemical reaction which releases oxygen and removes carbon dioxide. One canister will last 45 min when the wearer does hard work. The mask is equipped with a speaking diaphragm and an automatic timing device which rings a bell at the end of a preset interval. When the bell rings the wearer knows that he should return to fresh air. The entire unit weighs only 13½ lb and is very compact. Two of these units are owned by the Division, and the men usually work in pairs wherever masks are needed.

Safety Harness

The harness used in Dallas is nothing more than the harness used on parachutes. It is a very stout, light-weight harness with straps across the back and chest and two straps between the legs. There is also a V-type strap that snaps to each side of the harness in front. A tow rope can be snapped to this strap at the top which has a metal ring for that purpose. The harness is connected together with snaps which makes it easy to put on and take off. The wearer's body will stay in an upright position when raised or lowered. The harness also provides a convenient way to either lower or raise the wearer in or out of a manhole or lift station.

Manhole Blowers

The Division owns several manhole blowers and they are used frequently in sewer cleaning work. It has been found that a man can enter most manholes without danger if a blower is used. Gas masks are normally used when entering large trunk sewers.

Two-way Radios

The two-way radio is one of the most important aids to the maintenance organization. Four emergency trucks and seven passenger cars are equipped with them. The Division also has a central dispatcher who receives all complaints and in turn relays them to the proper personnel in the field. The radio-equipped passenger cars are used by the general foremen and other key personnel.

Sewer Maps

The value of sewer maps to a maintenance program is incalculable. In the old days, the sewer maintenance man remembered where all the sewer mains and manholes were located. He often knew where to find the sewer trouble in many sewer mains from past experience. However, today cities are too large to depend on such a vulnerable, transitory system.

The maps used in Dallas are drawn to a scale of 1 in. equals 400 ft, and they show the size of the pipe, direction of flow, and existing manholes and cleanouts. There is a key number on each main which refers to an as-built drawing which is stored in the engi-

* Manufactured by Mine Safety Appliance Co.

FIGURE 3.—Rocks removed from sanitary sewers in Dallas.

neering department. This plan shows all the details of the main such as date of construction, kind of pipe, kind of embedment, location ties, and profile. The as-built plan, however, is not used very often by the maintenance force because the 400-ft scale map usually serves their needs. The emergency crews, cleaning crews, and key personnel carry a full set of these small-scale maps with them at all times. They are used also by the engineering department as an aid to permit clerks and other personnel dealing with build-

FIGURE 4.—Chunks of grease removed with bucket machines.

ers, developers, and the public in general.

Sewer Maintenance Equipment

In Dallas, all city-owned automobiles, trucks, and other equipment, with the exception of sewer-cleaning equipment, are maintained by the municipal garage which is a separate city division. The equipment is leased to the Sewer Maintenance Division on a mileage basis for automobiles and trucks and on an hourly basis for other equipment. Almost any kind of equipment can be supplied by the garage. All divisions are assigned certain pieces of equipment which are used by that division only. This assures each division that its equipment will be available when needed. Automobiles, trucks, back-hoes, ditching machines, draglines, air compressors, concrete mixers, and various other equipment are available at the garage.

Sewer Cleaning Equipment

Sewer cleaning in Dallas started in 1950. At that time, one sewer rodding machine and one set of bucket machines (Figures 3 and 4) were purchased. This equipment was used to clean only sewers which were giving real trouble; no thought was given to a routine cleaning program. In 1953, the sewer-cleaning organization expanded and it continues to expand. Now, the Division owns two large rodding machines, five sets of bucket machines, and several small 2-hp rodding machines. Each year about 300 miles of sewers are cleaned. At present an attempt is being made to set up a long-range, routine cleaning schedule which will some day cover the entire sewer system.

SPLIT CHLORINATION *

By John M. Sherbeck

Superintendent, Sewage Treatment Plant, Bay City, Mich.

Compared with effluent chlorination, split chlorination or pre- and post-chlorination seems to be a more efficient disinfecting practice. In addition, the method is beneficial in other ways: it is very effective in reducing odors; it makes the skimming of fats and oils easier to accomplish; and it can enhance sedimentation by preventing gas formation during long retention periods.

The Bay City sewage treatment works is of the primary type with an average dry weather flow of from 6 to 8 mgd. The plant consists of three circular settling tanks and a circular chlorine contact tank. Each of these tanks has a capacity of 350,000 gal. When the flow is 8 mgd, the chlorine contact time is approximately 60 min; this helps to account for the large reduction in coliform organisms.

The most probable number of coliform organisms in Bay City's raw, unchlorinated sewage varies from about 1,600,000 to 24,000,000 per 100 ml. These values come from the results of 20 confirmed tests run according to "Standard Methods" (1).

Post-Chlorination

In 1955, between May 15 and Sept. 15, coliform tests were performed on

* Presented at the 34th Annual Meeting, Michigan Sewage and Industrial Wastes Assn.; Bay City, Mich.; May 11–13, 1959.

TABLE I.—Summary of Chlorination Results at Bay City, Mich.

Item	Single-Shot Chlorination		Split Chlorination	
	1955	1956	1957	1958
Sewage pumped (mil gal)	750	975	1,340	829
Chlorine used (lb)	75,300	69,200	99,800	84,700
Chlorine dosage (mg/l)	12.0	8.4	6.1	12.2
No. of samples	39	46	43	41
No. of samples with MPN/100 ml of:				
0 to 100	26	30	35	38
100 to 1,000	6	7	6	1
More than 1,000	7	9	2	2

39 chlorinated effluent samples (Table I). Of these, 26 had MPN's of less than 100 organisms/100 ml; of the remainder, 6 MPN's were between 100 and 1,000, and the other 7 were between 1,000 and 16,800. In that summer, 75,300 lb of chlorine was used to treat 750 mil gal (12.0 mg/l) to maintain a residual of from 0.5 to 1.0 mg/l. Chlorination was added only at the chlorine contact tank.

In 1956, with single-shot chlorination again, 68,200 lb of chlorine was used to treat 975 mil gal (8.4 mg/l). The results of coliform tests on 46 effluent samples showed that 30 had MPN's below 100 organisms/100 ml, 7 showed MPN'S between 100 and 1,000, and 9 MPN's exceeded 1,000.

Split Chlorination

In 1957 split chlorination was adopted. Chlorine was added on the discharge side of the raw sewage pumps, and a trace of residual was maintained at the weir of the clarifiers. Additional chlorine was then added at the chlorine tank to maintain a residual of from 0.5 to 1.0 mg/l. In that year two chlorinators were used, and 99,750 lb of chlorine was added to 1,340 mil gal for a dosage of 6.1 mg/l. The results of the coliform tests on 43 samples were as follows: in 35, the MPN was less than 100 organisms/100 ml; in 6, it was between 100 and 1,000; and in 2, it was over 1,000. Note that in the combined system as pumpage goes up the chlorine demand goes down.

Approximately ¾ of the chlorine was added before settling and ¼ was added at the chlorine retention tank.

In 1958 split chlorination was used again, but this time with only one chlorinator because of the high cost of water needed to operate two machines. Chlorine was added to the raw sewage until a trace residual appeared at the weir of the clarifiers, then by manipulation of valves, additional chlorine was added to the retention tank. The results from coliform tests on 41 samples showed that 38 had MPN values below 100 organisms/100 ml; 1 was between 100 and 1,000, and the remaining 2 were 1,100.

All results are compared in Table I.

Conclusions

It is believed that split chlorination is superior to one-point application. Lower numbers of surviving coliform organisms and more efficient use of chlorines are two results. At present, no statistical accounting can be given for the claims of odor control and improved settling, nonetheless, some improvement is apparent.

The only improvement lacking at Bay City is a system for adding chlorine farther upstream so that odors in the wet well can be controlled.

Reference

1. "Standard Methods for the Examination of Water, Sewage, and Industrial Wastes." 10th Ed., Amer. Pub. Health Assn., New York (1955).

COMMINUTOR MAINTENANCE *

By N. S. Bubbis

General Manager and Chief Engineer, Greater Winnipeg Sanitary District, Winnipeg, Manitoba

Normal domestic sewage contains rags and coarse solids in addition to regular household wastes. Discharges from industries and storm water, in the case of combined sewers, permit additional quantities of coarse solids to enter the sewer system. These solids cause the clogging of pumps and other equipment and interfere with the treatment processes. Consequently, it has been almost universal practice to install either manually or mechanically raked bar screens close to the point where sewage enters the treatment plant. The screenings have to be disposed of by removal and burning or burial, or by grinding and returning them to the sewage flow. While such installations are practical at the treatment plant, they are not very suitable for small pumping stations where the smaller pumps are particularly susceptible to clogging and where screens would constitute a messy and insanitary solution. In answer to this problem, the comminutor was developed.

The comminutor is a machine which cuts and shears waterborne solids into pieces small enough to pass through the slots of a rotating drum. The original machine, as developed by the Chicago Pump Co., is installed in a specially constructed basin with the flow passing into the drum and thence down through a discharge port. Several years ago, following the expiration of certain patent rights, two other firms developed modifications of the comminutor. The Worthington Pump Co. makes a similar unit called a comminutor, and Infilco Inc. makes another variation called the griductor; both can be placed in the regular channel without any special basin. More recently, the Chicago Pump Co. has developed a more radical modification called a barminutor which can also be installed directly in a flow channel. Recently, Infilco has developed a new type of machine called a "Rotagrater."

Winnipeg Operating Conditions

The Greater Winnipeg area is very flat and, consequently, a large number of low-lift pumping stations had to be constructed to intercept the sewage from the many trunk sewers. Comminutors were placed in 24 such stations constructed in 1937. There are now 31 pumping stations with 37 comminutors installed.

Conditions prevailing in Winnipeg are probably as severe as any likely to be encountered. First, the trunk sewers intercepted are all of the combined type and they were laid on very flat grades. Second, the long and severe winters require the use of large quantities of sand, cinders, and finely crushed stone for ice control on the streets. Most of this material is later flushed to the sewers in the spring with the winter accumulation of some 50 in. of snow. As a result, the amount of comminutor maintenance required is much heavier than that normally required by units used in more temperate locales.

There are two conditions in Winnipeg which result in a somewhat unusual approach to the maintenance program. These are: (*a*) the cost of replacement parts is unusually high because of import duties and until very recently, a 10-per cent federal sales tax, and (*b*) the city of Winnipeg engineering department operates a first-class machine shop which economically manufactures hydrants, corporation and curb cocks, meter gear trains, etc. As a result, the district has been able to

* Presented at the 32nd Annual Meeting, Federation of Sewage and Industrial Wastes Assns.; Dallas, Tex.; Oct. 12–15, 1959.

make its own modifications to the comminutors and manufacture certain spare parts.

Maintenance

The comminutor is primarily a mechanical cutting and screening device, and, while it is very sturdy in construction, its cutting action works within very close limits. It is a precision machine which has to be well maintained to function properly. An iron casting forms the main framework for the slotted drum which is rotated at about 58 rpm. On the drum, small projections or 'teeth' protrude at different points throughout its depth. These teeth are so placed that they mesh with a fixed comb on the cast iron framework. Also on the drum, at intervals, there are shear bars which shear and cut the solids at the comb. The action is such that the flow along the channel in which the comminutor is installed brings the sewage to the face of the drum and the solids are then cut by either the teeth or the shear bars as the sewage tries to pass through the slotted drum.

Excessive Wear and Corrosion

One of the main problems encountered after the initial installation was the excessive wear of the combs and shear bars which were originally made of mild steel and tipped with Stellite, a non-ferrous hard metal, to resist abrasion. These difficulties were overcome by using stainless steel instead of mild steel for the combs and shear bars. The combs and shear bars, however, had to be grooved so that Stellite would bond properly. The teeth are made of Stellite and are manufactured in the city's machine shop.

Another early problem was the excessive wear on the drums caused by heavy grit and, partly, by rusting. The greatest wear took place on the bottom slots, decreasing progressively to the top. In view of this it was decided to split the 15- and 25-in. drums horizontally into two sections. By doing this, a bottom section showing signs of excessive wear could be replaced at about one-half the cost of a new drum. After some experimentation, it was decided to use the International Nickel Company's 'Ni-resist,' a very hard metal, instead of the cast steel originally supplied. All drums are now made of this material and their service life has been increased approximately four- or five-fold. In some instances, top sections have been in continuous service for approximately 10 years. Records show that at the end of 1958 one 15-in. machine had been operating for 722 days; one 10-in. machine has been in continuous operation for 1,582 days; and most of the 25-in. machines are operated for from 400 to 500 days before new bottom sections are needed.

Lubrication

Lubrication is a very important part of any maintenance program. Service crews check the grease level of the comminutors each weekday. It is believed, just as in the case of pump bearings, that sewage with its entrained grit must be kept away from rotating bearings and this can be done best by regular checking and frequent greasing. A good waterproof, mineral-base bearing grease is used. The motor and gear reducer are also checked periodically and lubricated. A preventive maintenance check chart is used by the service crews.

Ragging

Another problem not entirely solved is ragging. The comminution of the rags in sewage permits them to pass through the slots of the drum. Some rags, however, wind around the bottom nut of the comminutor drum or some other projection and a rag rope is formed which is ultimately dislodged by sheer weight after a period of time. For this reason, the inside of all the drums are smoothed off and the bottom nuts capped. This step reduced the amount of ragging, but it did not eliminate the problem entirely.

TABLE I.—Comminutor Maintenance Costs

Year	Number of Units in Operation			Number of Units Changed			Cost of Repairing and Overhauling Machines		
	10-in.	15-in.	25-in.	10-in.	15-in.	25-in.	10-in.	15-in.	25-in.
1955	5	17	5	2	14	2	$465	$3,315	$3,459
1956	6	19	5	2	18	2	533	3,642	1,790
1957	7	19	5	3	20	3	432	4,762	2,965
1958	7	19	6	3	22	2	948	4,638	2,178

Frequent cleaning of the comminutor, the basin, and all parts is another deterrent to the formation of rag ropes, and it helps to eliminate the abrasive action of grit. To make flushing as easy as possible, hose bibs have been installed in all stations.

Method of Installation

One other point should be mentioned regarding installation of comminutors: with the large machines, a fabricated steel ring which fits around the base of the machine is used instead of anchor bolts. This ring permits a tight fit and is held in place only by the concrete projection around the cast iron framework of the machine. This makes it possible to remove a 25-in. machine very rapidly. On the small machines, i.e., 15- and 10-in., anchor bolts are used.

Maintenance Personnel and Costs

Initially, it was planned that a single three-man crew would take care of the District's 24 automatically operated comminutor-pumping stations. This was not found practical. The District's 20 miles of intercepting sewers, 10 river crossings, 31 comminutor-pumping stations, and 27 floodwater pumping stations are now maintained by a staff of 10 men and a supervisor. Two three-man service crews are used for the daily visits to each station and they carry out routine maintenance of all equipment. The maintenance and repair crew consisting of four men has a hoist-equipped truck and other tools required to perform major tasks. All trucks are equipped with two-way radios.

As mentioned earlier, all repair work on the comminutors, including overhauling and rebuilding as well as the manufacture of new combs, shear bars, drums, and teeth, is carried out at the city machine shop. This work keeps one regular employee occupied for approximately two-thirds of his time. The cost of recent work done by the machine shop and the number of machines changed each year are listed in Table I. It is not possible to give the portion of the cost chargeable to comminutor maintenance alone, but the total cost for the maintenance and operation of the collection system amounted to $103,786 for 1958.

The District, as it does with all its equipment, maintains an adequate supply of spare parts. Based on past experience, one spare comminutor is stocked for every five in service; also, several gear reducers and one or two spare motors are stocked, although over a 20 year period, motors and gear reducers have had to be replaced in only one or two instances. During the 1950 flood, 15 motors were submerged before they could be removed. Fortunately, one or two days' baking was all that was required to restore them to usable condition.

Conclusion

The effectiveness of any maintenance program is measured by two factors: (*a*) the unit cost of maintenance, and (*b*) the amount of equipment downtime. The District feels that it has a fair record on both counts, bearing in mind the difficult operating conditions.

TIPS AND QUIPS

Rubber Pipe

One of the largest installations of its kind was recently completed in New Jersey when an 1,864-ft length of large-diameter rubber pipe * was laid from the shore to the channel of the lower Delaware River. The pipe was designed to discharge processing wastes from a large chemical plant.

FIGURE 1.—Workmen prepare to connect two lengths of rubber pipe. Note heavy, cast iron collars which anchored pipe to river bottom.

The line was installed from an offshore dredge which first scooped out a trench on the river bottom. Successive 50-ft flanged lengths were bolted together, using corrosion-resistant bolts and nuts. Flexibility of the line simplified the bolting operations. The free end was just lifted out of the water, without disturbing the sections already in place, and attached to the next length on the dredge. Then the dredge backed off one pipe length and dropped the newly attached length in place (Figures 1 and 2). A heavy cast iron collar bolted to each rubber pipe anchored it to the bottom.

Operated as an open-end 12-in. diameter pumpline, the rubber pipe is designed to withstand 100 psi. This strength is provided by a nylon and steel wire carcass. A neoprene cover is used for protection against external damage. Depending on the tides, 50 to 80 ft of the line could be exposed to weathering and floating debris.

FIGURE 2.—A 50-ft section of pipe is being lifted into position for attachment to an 1,800-ft waste line in the Delaware River.

* Manufactured by Raybestos-Manhattan, Inc., Passaic, N. J.

Don't Lose Your Head †

At the Kodak Park Works of the Eastman Kodak Company in Rochester, N. Y., the waste treatment plant is used to generate electricity. Now a lot of plants generate power in one way or another, but what makes Eastman's plant unique is that the electricity is generated in a hydroelectric turbine. Wastes on their way to the treatment plant drop 120 ft, furnishing the necessary energy to generate enough electricity to meet most plant needs and, at times, produce a surplus!

† From *Plant and Power Services Engineer* (Dec. 1959).

Football Game Aids Gas Production

Dave Backmeyer writes that for the 20-day period ending Dec. 31, 1959, the average daily gas production at the Miami, Fla., sewage treatment plant was 260,000 cu ft. The Orange Bowl football game and parade on New Year's Day always bring thousands of visitors to downtown Miami. On Monday following New Year's week end, the gas production at the plant was the greatest for any single day on record. A total of 390,800 cu ft was metered in the 24-hr period.

Operators with indifferent digesters please note.

Can't Keep Them down on the Farm

More and more agricultural materials enter the manufacturing chain and wind up in some unrecognizable form. A partial listing of some of the products is shown below. Many of these have associated with them wastewaters which require treatment.

Agricultural Materials and End Products

Agricultural Raw Material	Used to Manufacture
(*a*) Carbohydrates	
cellulose from	
cotton linters, wood pulp	nitrocellulose, rayon, lacquers, cellophane, acetate, paper
starch (corn)	adhesives, dextrose, sorbitol
corn steepwater	penicillin
oat hulls, corn cobs, bagasse	furfural, nylon, medicinals
sugars, molasses	allyl sucrose, alcohol, acetone
lignin	vanillin, leather, textile chemicals
rosin	soaps, paints, plastics, paper chemicals
lactose (milk sugar)	pharmaceuticals
lactic acid	food, pharmaceuticals
(*b*) Fats and Oils	
cottonseed oil	margarine, shortening
soybean oil	paints, shortening, lecithin, margarine
tung oil	paints, varnish
peanut oil	pharmaceuticals
linseed (flax) oil	paints, printing inks
castor oil	pharmaceuticals, paints
tall oil	paints, soap, detergents
inedible tallow and greases	soap, glycerine, red oil, lubricants
(*c*) Protein	
sugar beet pulp, wheat gluten	food flavor intensifiers
milk	casein adhesives, coatings
blood	adhesives
(*d*) Other	
buckwheat	pharmaceuticals
alfalfa	chloresium for wound healing, odor reduction
tobacco	insecticides
turpentine	paints, plastics
animal organs	ACTH, cortisone, pepsin extracts, insulin
bone	gelatins, bone ash

A. R. Strate and greenhouse

Alf Strate, Superintendent of the Nampa, Idaho, sewage treatment plant, has been hoping for a long time to build a greenhouse at the plant. The opportunity came when used building materials from another city project became available. Heated by waste gas, the 20- by 30-ft greenhouse produces flowers for other city departments.

News and Notes

Of Persons and Activities in the Wastewater Field

John R. Hartley was elected President of Builders-Providence, Inc., Division of **B-I-F Industries.** Mr. Hartley joined the firm in 1929 and has served, successively, as apprentice draftsman, sales engineer, manager of project sales, and general manager.

Arthur H. Niles, Treasurer of the **Water Pollution Control Federation,** has retired as Commissioner of Sewage Disposal for the city of Toledo, Ohio. He will make his new home at 433 N. E. Thirty-Third St., Boca Raton, Fla., where he will be available as a consulting engineer.

Camp, Dresser, and McKee, consulting engineers, have moved their office from 6 Beacon St. to 18 Tremont St., Boston 8, Mass.

John R. Thoman was named the Regional Program Director, for the Division of Water Supply and Pollution Control of U. S. Public Health Service's Atlanta, Ga., office.

Clarence W. Bogenhagen died suddenly on Feb. 11, 1960. Mr. Bogenhagen was superintendent of the Fond du Lac, Wis., sewage treatment works and served as Secretary-Treasurer to the Conference of Wisconsin Sewage Works Operators.

Williams, Hatfield and Stoner, Inc., civil engineers, have moved their office to 2312 Wilton Dr., Fort Lauderdale, Fla.

Richard F. Poston is now the Public Health Service Consultant to the U. S. Study Commision Southeast River Basins, Atlanta, Ga.

Worthen H. Taylor has been appointed Director of the Division of Sanitary Engineering of the Massachusetts Department of Public Health.

Earnest Boyce, Chairman of the Department of Civil Engineering, University of Michigan, and **Federation Past-President,** will serve as a member of a three-man team of engineers chosen to study sanitation and water supply problems in villages of India. The two other team members are **Morton W. Lieberman,** formerly of Tippetts, Abbet, McCarthy, and Stratton, and **Dwight D. Metzler** of the Kansas State Board of Health.

Along with Indian government representatives, the team will make two to three week inspection trips to the east, south, and west portions of India. After the trips, they will return to New Delhi to prepare reports and make recommendations for a third five-year plan.

Paul Eastman has been transferred from Washington, D. C., to become the Regional Program Director for the Division of Water Supply and Pollution Control of the U. S. Public Health Service's San Francisco office.

George A. Wesson, Jr., has joined the research, engineering, and development staff of the **American Well Works,** Aurora, Ill.

Joseph W. Lipscomb recently became the Chief Engineer of the Florida office of **Gannett Fleming Corddry and Carpenter, Inc.** The firm's Florida office is now located at 523 N. Halifax Ave., Daytona Beach, Fla.

Walter A. Gardner has been appointed Vice-President in charge of engineering for the **Harrison Engineering Company,** Gastonia, N. C.

Paul W. Reed is now the Public Health Service Consultant to the U. S. Study Commission, Texas River Basins, Houston, Tex.

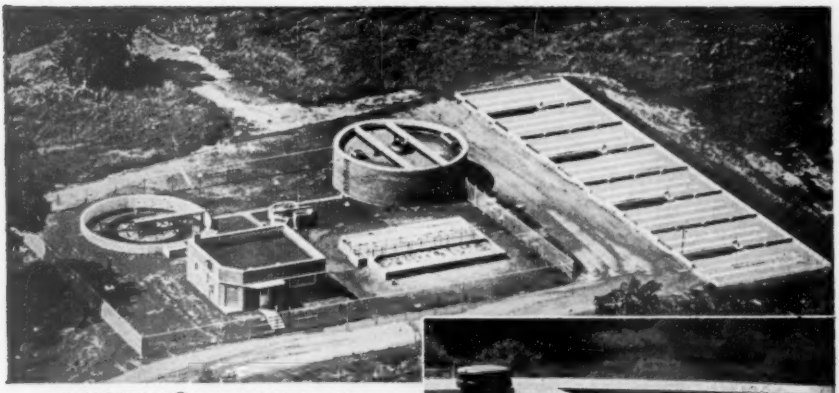
George W. Kelsey was elected Senior Vice-President of **B-I-F Industries** at a recent meeting of the Board of Directors. Mr. Kelsey also serves as President of the Water and Sewage Works Manufacturers Association.

Bowe, Albertson & Associates, engineering firm of New York, has opened a New England office at 1000 Farmington Ave., West Hartford, Conn.

Ray Raneri, prominent U. S. Public Health Service Engineer, died Feb. 25, 1960. Mr. Raneri was well known for his work in connection with the planning and operation of the Robert A. Taft Sanitary Engineering Center in Cincinnati, Ohio.

Boyer & Sweeney, engineers of Salina, Kans., announced their new firm name of **Boyer-Sweeney-Hagadorn Engineers.**

Joseph V. Smith has been named manager of the South Central Sanitary Sales Division of **Dorr-Oliver Inc.** Mr. Smith makes his headquarters at 4117 Broadway, Kansas City, Mo.



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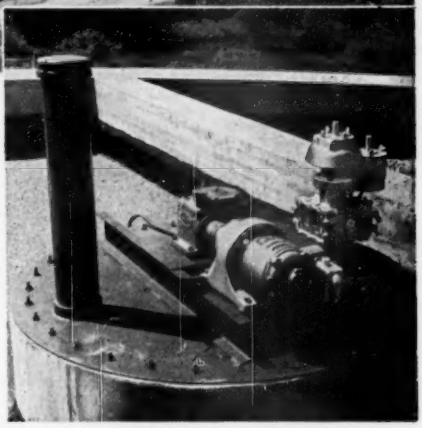

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American Well Works
Ralph B. Carter Co.
Dorr-Oliver Inc.
Dow Chemical Co.
Eimco Corp.
Filtration Equipment Corp.
Industrial Materials Co.
Infilco Inc.
Keasbey & Mattison Co.
Komline-Sanderson Engr. Corp.
Lakeside Engineering Corp.
Nichols Engr. & Research Corp.
Pacific Flush Tank Co.
Smith & Loveless, Inc.
Trickling Filter Floor Inst.
Walker Process Equipment Inc.
Yeomans Brothers Co.
Zimmer & Francescon

Filter Materials
Carborundum Co.
Filtration Equipment Corp.
Graver Water Conditioning Co.
Industrial Materials Co.
Johns-Manville Sales Corp.
Vulcan Materials Co.

Filters, Diatomaceous Earth
Graver Water Conditioning Co.
Yeomans Brothers Co.

Filters, Vacuum (*also see* Vacuum Filters)
Bird Machine Co.
Dorr-Oliver Inc.
Eimco Corp.
Proportioneers Div., B-I-F Industries

Flocculating Equipment
Alpha Ltd. (Switzerland)
American Well Works
Ralph B. Carter Co.
Chain Belt Co.
Dorr-Oliver Inc.
Eimco Corp.
Graver Water Conditioning Co.
Hardinge Co., Inc.
Infilco Inc.
Jeffrey Mfg. Co.
Lakeside Engineering Corp.
Link-Belt Co.
Stuart Corp.
Walker Process Equipment Inc.
Webster Mfg., Inc.
Zimmer & Francescon

Flotation Equipment
Chain Belt Co.
Dorr-Oliver Inc.
Eimco Corp.
Graver Water Conditioning Co.
Komline-Sanderson Engr. Corp.
Yeomans Brothers Co.

Flow Measurement
Bailey Meter Co.
Burgess-Manning Co., Penn Instruments Div.
Filtration Equipment Corp.

Gas Control Equipment
Alpha Ltd. (Switzerland)
Burgess-Manning Co., Penn Instruments Div.
Ralph B. Carter Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
Foxboro Co.
Homestead Valve Mfg. Co.
Pacific Flush Tank Co.
Rockwell Mfg. Co.
Vapor Recovery Systems Co.
Walker Process Equipment Inc.
Zimmer & Francescon

Gas Diffusers (*see* Diffusers, Gas)

Gas Holders, Boilers, and Appurtenances
Ralph B. Carter Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
Johns-Manville Sales Corp.
Pittsburgh-Des Moines Steel Co.
Walker Process Equipment Inc.

Gaskets
Johns-Manville Sales Corp.
Keasbey & Mattison Co.

Gates
Armco Drainage & Metal Products, Inc.
Filtration Equipment Corp.
Industrial Materials Co.
Snow Gates & Valves, Inc.
Webster Mfg., Inc.

Generators
Climax Engine Mfg. Co.
Fairbanks, Morse & Co.
General Electric Co.
Westinghouse Electric Corp.
Worthington Corp.

Grease Control Materials
Cloroben Chemical Corp.

Grinders (*also see* Shredders and Grinders)
Alpha Ltd. (Switzerland)
American Well Works
Chain Belt Co.
Dorr-Oliver Inc.
Gruendler Crusher & Pulverizer Co.
Jeffrey Mfg. Co.
Yeomans Brothers Co.

Grit Collection and Conditioning Equipment
Alpha Ltd. (Switzerland)
American Well Works
Chain Belt Co.
Chicago Pump Co.
Dorr-Oliver Inc.
Eimco Corp.
Infilco Inc.
Jeffrey Mfg. Co.
Link-Belt Co.
Stuart Corp.
Walker Process Equipment Inc.
Webster Mfg., Inc.
Zimmer & Francescon

New Journal Index Ready!

SEWAGE AND INDUSTRIAL WASTES

INDEX

Volumes 21-30, 1949-1958

The Ten-Year Index to SEWAGE AND INDUSTRIAL WASTES covering the period of 1949–1958 is now available. The 168-page volume is offered in a handsome maroon buckram binding at $4.00 and in an attractive heavy paper cover at $3.00.

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Morse Boulger, Inc.
Nichols Engr. & Research Corp.
Pittsburgh-Des Moines Steel Co.
Walker Process Equipment Inc.

Ion-Exchange Equipment
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Insect Control
Cloroben Chemical Corp.
Leeco Chemical Co., Div. of Leeco Gas & Oil Co.

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Centriline Corp.
Industrial Pipe Repair Corp.
National Water Main Cleaning Co.

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Bailey Meter Co.
Builders-Providence Div., B-I-F Industries
Burgess-Manning Co., Penn Instruments Div.
Fischer & Porter Co.
Fisher Scientific Co.
Foxboro Co.
General Electric Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co., Brown Instruments Div.
Rockwell Mfg. Co.
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Westinghouse Electric Corp.

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Gray Concrete Pipe Co., Inc.
Keasbey & Mattison Co.
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Perry-Austen Mfg. Co.
Stebbins Engr. & Mfg. Co.
Wedge-Lock Clay Pipe Mfrs.

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Smith-Blair, Inc.
U. S. Pipe & Foundry Co.
Wedge-Lock Clay Pipe Mfrs.

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Fisher Scientific Co.
General Chemical Div., Allied Chemical Corp.
Stuart Corp.

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Smith & Loveless, Inc.
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Westinghouse Electric Corp.

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Vapor Recovery Systems Co.
Vulcan Materials Co.

Masonry Building Materials
American-Marietta Co.
Portland Cement Assn.
Price Bros. Co.
Vulcan Materials Co.

Meter Boxes
Johns-Manville Sales Corp.

Meters (Sewage, Sludge, Water, Air, and Gas)
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Builders-Providence Div., B-I-F Industries
Burgess-Manning Co., Penn Instruments Div.
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Filtration Equipment Corp.
Foxboro Co.
Infilco Inc.
Minneapolis-Honeywell Regulator Co., Brown Instruments Div.
Rockwell Mfg. Co.
Roots-Connersville Blower, Div. of Dresser Industries Inc.
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Eimco Corp.
Infilco Inc.
Jeffrey Mfg. Co.
Walker Process Equipment Inc.
Wemco Div., Western Machinery Co.

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General Electric Co.
Link-Belt Co.
Westinghouse Electric Corp.
Worthington Corp.

Odor Control Materials
Airkem Inc.
Cloroben Chemical Corp.
Florasynth Laboratories, Inc.
Tennessee Corp.
Wallace & Tiernan Inc.

Odor Counteractants
Airkem Inc.

Package Treatment
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Chicago Pump Co.
Dorr-Oliver Inc.
Infilco Inc.
Municipal Service Co.
Smith & Loveless, Inc.
Yeomans Brothers Co.

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Koppers Co., Inc.
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Stebbins Engr. & Mfg. Co.

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Industrial Materials Co.
Johns-Manville Sales Corp.
Keasbey & Mattison Co.

Pipe, Cast Iron
American Cast Iron Pipe Co.
Cast Iron Pipe Research Assn.
Industrial Materials Co.
U. S. Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Clay
Industrial Materials Co.
National Clay Pipe Mfrs., Inc.
Wedge-Lock Clay Pipe Mfrs.

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Centriline Corp.
Cloroben Chemical Corp.
Flexible Inc.
Homestead Valve Mfg. Co.
National Water Main Cleaning Co.
W. H. Stewart, Inc.

Pipe Coatings and Linings
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American Pipe & Construction Co.
Centriline Corp.
Koppers Co., Inc.
National Water Main Cleaning Co.
Perry-Austen Mfg. Co.
Stebbins Engr. & Mfg. Co.
Vulcan Materials Co.

Pipe, Concrete
American Concrete Pressure Pipe Assn.
American-Marietta Co.
American Pipe & Construction Co.
Gray Concrete Pipe Co., Inc.
Lock Joint Pipe Co.
Portland Cement Assn.
Price Bros. Co.
Vulcan Materials Co.

Pipe, Fiber
Brown Co.

Pipe Fittings
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Cast Iron Pipe Research Assn.
Keasbey & Mattison Co.
Price Bros. Co.
Smith-Blair, Inc.
U. S. Pipe & Foundry Co.
Wedge-Lock Clay Pipe Mfrs.
R. D. Wood Co.

Pipe Jointing Materials (*see* Jointing Materials)

Pipe, Plastic
Amercoat Corp.
Evanite Plastic Co.

Pipe, Repairs
Centriline Corp.
Industrial Pipe Repair Corp.
Smith-Blair, Inc.

Pipe, Steel
American Pipe & Construction Co.
Armco Drainage & Metal Products, Inc.
Keasbey & Mattison Co.

Plastic Pipe Products
Amercoat Corp.
Evanite Plastic Co.
Keasbey & Mattison Co.

Publications
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John Wiley & Sons, Inc.

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Fischer & Porter Co.
Foxboro Co.
General Electric Co.
Minneapolis-Honeywell Regulator Co., Brown Instruments Div.
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Worthington Corp.
Zimmer & Francescon

Proceedings of Member Associations

NEBRASKA SEWAGE AND INDUSTRIAL WASTES ASSOCIATION

The 8th Annual Meeting of the Nebraska Sewage and Industrial Wastes Association was held at the Fort Sidney Motor Hotel, Sidney, Nebr., Nov. 5–6, 1959.

Operating problems received generous attention at the technical sessions:

"Sewage Pumps, Their Maintenance and Application," by Frank A. Nolte, Manager, Pump Department, Fairbanks, Morse and Co., Omaha, Nebr.

"Cost of Electrical Power at Nebraska Sewage Treatment Plants," by V. J. Lechtenberg, Omaha, Nebr., and W. F. Rapp, Jr., Lincoln, Nebr.

"Values and Benefits of Keeping Operating Records at Sewage Treatment Plants," by Thomas Cooper, Chicago Pump Co., Chicago, Ill.

"Sewage Lagoon Maintenance Problems," by W. F. Rapp, Jr., Nebraska Health Department, Lincoln, Nebr.

"Economical Painting at Sewage Plants," by Lew G. Salisbury, Inertol Co., Inc., Kansas City, Mo.

"Weed Control at Sewage Plants and Along Outfall Ditches," by Vern Anderson, Geigy Chemical Co., Kansas City, Mo.

"A History of Sewage Treatment in Sidney," by Floyd Sanks, Water and Sewer Commissioner, Sidney, Nebr.

Officers elected to serve during 1959–60 are:

President: V. J. Lechtenberg, Omaha.
Vice-President: Walter Arfmann, Cozad.

(*Continued on page 185a*)

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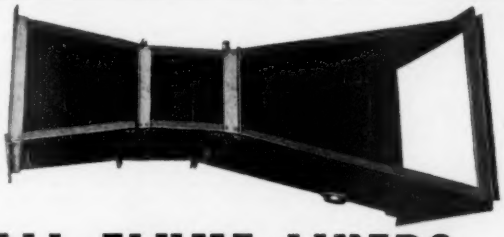

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Pumps, Grit
Nagle Pumps, Inc.

Pumps, Sewage
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Gorman-Rupp Co.
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Marlow Pumps, Div. of Bell & Gossett Co.
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Wallace & Tiernan Inc.
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Wallace & Tiernan Inc.
Wemco Div., Western Machinery Co.
Worthington Corp.
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Eimco Corp.
Fairbanks, Morse & Co.
Gorman-Rupp Co.
Marlow Pumps, Div. of Bell & Gossett Co.
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Minneapolis-Honeywell Regulator Co., Brown Instruments Div.
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Stebbins Engr. & Mfg. Co.

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Vulcan Materials Co.

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Eimco Corp.
Infilco Inc.
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Link-Belt Co.
Walker Process Equipment Inc.
Zimmer & Francescon

Sedimentation Equipment (*also see* Clarifier Equipment)
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Dorr-Oliver Inc.
Eimco Corp.
Graver Water Conditioning Co.
Hardinge Co., Inc.
Infilco Inc.
Jeffrey Mfg. Co.
Komline-Sanderson Engr. Corp.
Lakeside Engineering Corp.
Link-Belt Co.
Walker Process Equipment Inc.
Webster Mfg., Inc.
Yeomans Brothers Co.

Sewer Cleaning Equipment
Flexible Inc.
Homestead Valve Mfg. Co.
National Water Main Cleaning Co.
W. H. Stewart, Inc.

Sewer Inspection (*see* Inspection, Sewers)

Shredders (*also see* Comminutors and Grinders)
Alpha Ltd. (Switzerland)
Chain Belt Co.
Dorr-Oliver Inc.
Gruendler Crusher & Pulverizer Co.
Jeffrey Mfg. Co.
Yeomans Brothers Co.

Siphons
Alpha Ltd. (Switzerland)
American Well Works
Ralph B. Carter Co.
Eimco Corp.
Infilco Inc.
Lakeside Engineering Corp.
Pacific Flush Tank Co.
Yeomans Brothers Co.

Sludge Concentrators
Eimco Corp.
Nichols Engr. & Research Corp.

Sludge Flotation Equipment (*see* Flotation Equipment)

Sludge Handling and Control
Chain Belt Co.
Dorr-Oliver Inc
Eimco Corp.
Jeffrey Mfg. Co.
Link-Belt Co.
Nichols Engr. & Research Corp.
Walker Process Equipment Inc.

Sludge Removal Equipment
Jeffrey Mfg. Co.
Link-Belt Co.
Webster Mfg., Inc.

Sludge Shredders
Alpha Ltd. (Switzerland)
Gruendler Crusher & Pulverizer Co.
Jeffrey Mfg. Co.

Sprockets
Chain Belt Co.
Link-Belt Co.
Webster Mfg., Inc.

Steel Pipe Products
Armco Drainage & Metal Products, Inc.

Structural Metal
Alpha Ltd. (Switzerland)

Switchgear
Alpha Ltd. (Switzerland)
General Electric Co.
Westinghouse Electric Corp.
Worthington Corp.

Tanks
Chain Belt Co.
Link-Belt Co.
Pittsburgh-Des Moines Steel Co.
Preload Co., Inc.
Stebbins Engr. & Mfg. Co.

Television, Closed Circuit
Industrial Pipe Repair Corp.

Tools
Flexible Inc.

Transformers
Alpha Ltd. (Switzerland)
General Electric Co.
Westinghouse Electric Corp.

Trickling Filter Equipment (*see* Filter Equipment, Filter)

Trucks and Tractors
Eimco Corp.

Turbines
Worthington Corp.

Vacuum Filters (*also see* Filters, Vacuum)
Bird Machine Co.
Dorr-Oliver Inc.
Eimco Corp.
Komline-Sanderson Engr. Corp.
Proportioneers Div., B-I-F Industries

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Armco Drainage & Metal Products, Inc.
Builders-Providence Div., B-I-F Industries
Chapman Valve Mfg. Co.
DeZurik Corp.
Filtration Equipment Corp.
Homestead Valve Mfg. Co.
Industrial Materials Co.
Iowa Valve Co.
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Snow Gates & Valves, Inc.
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W-K-M Division, ACF Industries, Inc.
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Yeomans Brothers Co.

Vessels (*see* Tanks)

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Fisher Scientific Co.
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Secretary-Treasurer: E. Bruce Meier, Omaha.

E. Bruce Meier
Secretary-Treasurer

NORTH DAKOTA WATER AND SEWAGE WORKS CONFERENCE

The 31st Annual Convention of the North Dakota Water and Sewage Works Conference was held at the Dacotah Hotel, Grand Forks, N. Dak., Sept. 23–25, 1959. More than 160 attended the 3-day session.

Papers of particular interest to those in the wastewater field included:

"Ten Years of Progress in the Red River Basin," by H. C. Clare, USPHS Regional Office, Kansas City, Mo.

"The Potato Industry in North Dakota," a panel discussion.

"Responsibility of Water and Sewage Works Superintendents in Service Connections," by E. Welch, State Plumbing Inspector, Bismarck, N. Dak.

"Water Development in North Dakota," by Bruce Johnson, Projects Manager, Bureau of Reclamation, Bismarck, N. Dak.

"Problems in Developing Municipal Facilities," by Alan Webster, City Manager, Grand Forks, N. Dak.

At the business meeting the following officers were elected for 1959–60:

President: Frank Orthmeyer, Bismarck.
Vice-President: Richard Fuller, Southwest Fargo.
Secretary-Treasurer: W. Van Heuvelen, Bismarck.

W. Van Heuvelen
Secretary-Treasurer

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Superintendent
Sewage Treatment Plant
Sarnia, Ontario

The City of Sarnia is constructing a 4 million dollar Sewage Treatment Plant. It will be ready to go into operation early this fall. We are looking for a good man to work on the final stages of construction and installation of equipment and to take over as Superintendent when the plant is completed. All inquiries will be treated as confidential. For further information, write:

Mr. H. G. McKittrick
Personnel Officer
City Hall
Sarnia, Ontario, Canada

NEW

MANUAL AVAILABLE

M. O. P. #9 — SEWER DESIGN AND CONSTRUCTION

Several years of joint effort between the Water Pollution Control Federation and the Sanitary Engineering Division of the American Society of Civil Engineers have produced a major addition to the "Manuals of Practice" series. "Design and Construction of Sanitary and Storm Sewers" has been designated M. O. P. #9.

This illustrated, 12-chapter manual of 283 pages includes detailed coverage of project planning, investigations, hydraulics, design, construction materials and methods, specifications, structural requirements of storm and sanitary sewers, and pumping stations.

A convenient order form for this and other publications appears elsewhere in this issue. The manual is $7 per copy postpaid if remittance is with order. Federation members may purchase the manual for $3.50.

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Coagulates over wide pH ranges and provides efficient operation regardless of rapid variations of raw sewage—Very effective for conditioning sludge prior to vacuum filtration or drying on beds.

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reason: recessed impeller; continuous open passage *(what will go in will come out)*

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Equipment and Supply Lines

Oil Separator—A new oil intercepting system is claimed to be particularly adaptable in garages, service stations, machine shops, etc., where it is desirable to reclaim oil.—Series GH Oil Separator, Josam Manufacturing Co., Dept. X-51, Michigan City, Ind.

Grouting—A 16-page bulletin explains how to avoid shrinkage in grouting machinery and equipment through the use of Embeco non-shrink grout. Information on recommended mixes are contained in Bulletin Eld.—The Master Builder Company, Cleveland 3, Ohio.

Package Plant—A new small sewage treatment plant, the "Sparjair," utilizes the contact stabilization process. Units come in sizes capable of treating wastes from as few as 50 people to as many as 5,000.—Bulletin 22-S-94, Walker Process Equipment, Inc., P. O. Box 266, Aurora, Ill.

Sludge Shredders—A new edition of "Sewage Sludge Utilization Datalog" is available. The new booklet reviews and analyzes current writing on the subject of sludge shredding.—Royer Foundry & Machine Company, 175 Pringle St., Kingston, Pa.

Compressors—Heavy-duty, oil-free compressors are described in Bulletin A-44. Compressors ranging in size from 96 to 939 cfm are discussed.—Joy Manufacturing Company, Oliver Building, Pittsburgh 22, Pa.

Wastes Treatment Equipment—Catalog 952 of the Jeffrey Manufacturing Company, Columbus 16, Ohio, contains sections on bar screens, grinders, collectors, settling tanks, scum removers, feeders, conveyors, bucket-elevators, chains, and sprockets.

Sump Pump Motor—A complete sump pump motor package with a factory-installed switch, cord, and plug set has been introduced by the General Electric Company.

Sewage Pump—Trash-sewage pumps featuring an open impeller and easily removed end plate are described in Bulletin 8-ST-11.—Gorman-Rupp Company, 305 Bowman St., Mansfield, Ohio.

Nozzles—A complete line of spray nozzles is described in a 32-page catalog of the Wm. Steinen Mfg. Co., Industrial Nozzle Division, 45 Bruen St., Newark 5, N. J.

DO Meter—Information on an automatic DO analyzer and recorder is available from the Hay Corporation, Michigan City, Ind. The unit is designed to measure DO in sewage, industrial wastes, streams, lakes, etc.

Ductile Iron—A new metal combining the corrosion resistance of cast iron with mechanical properties similar to those of steel is being used in a new line of pipe, fittings, and specials. A catalog describing the new line is available from the American Cast Iron Pipe Company, Birmingham 2, Ala.

Ditcher—A new, medium-duty ditcher features a torque-converter drive and complete hydraulic action (Figure 1). The ditcher digs up to 7 ft, with widths of cut up to 44 in. It is transportable on standard trailers without special teardown.—Model 318, Garwood Industries, Inc., Wayne, Mich.

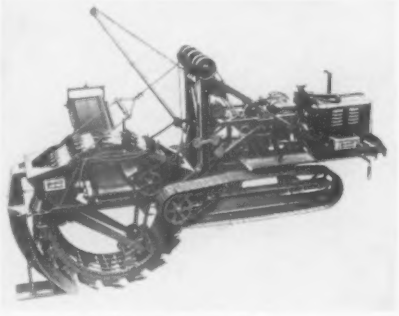

FIGURE 1.—Ditcher.

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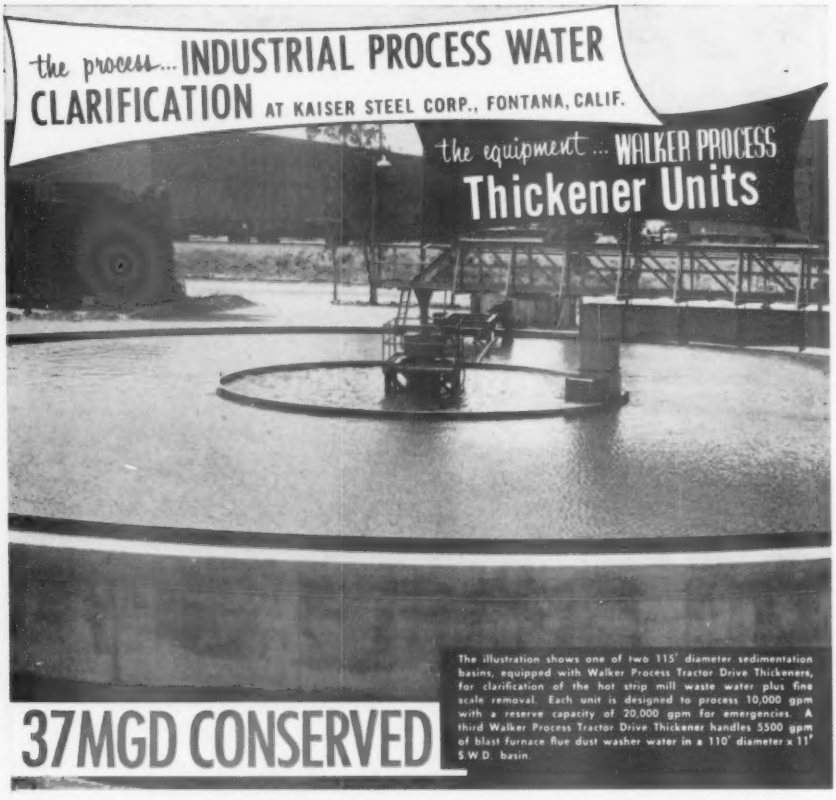
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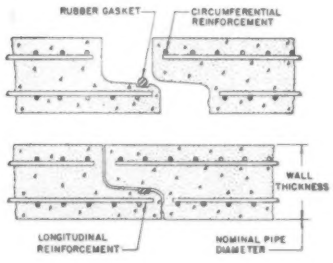

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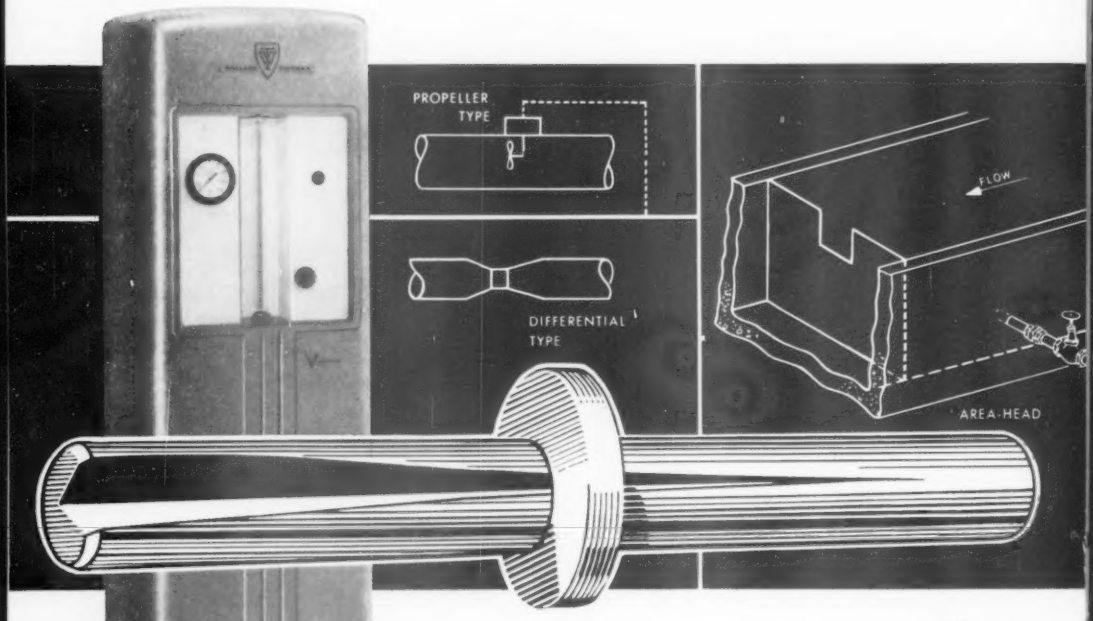
ready and waiting for your signal

Lots of jobs are done better automatically. Changing chlorine feed rate is now one of them. With the V-notch...the straight line metering action of the plug through the ring makes automation simple.

Merely by adding, not adapting, a W&T V-notch Chlorinator can give you any degree of automatic control you want—step rate, start-stop, program, proportional, even residual control.

With a V-notch Chlorinator any signal from any primary metering device can control chlorine feed. So anyone can afford automation.

And, of course, the right plastics make the whole chlorinator chlorine-proof.



A booklet, "The V-notch Story" will tell you about all the W&T V-notch Chlorinator features. For your copy write Dept. S-135.84.

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